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ALAMEDA POINT
SSIC NO. 5090.3

Southwest Division
Naval Facilities Engineering Command
Contracts Department
1220 Pacific Highway, Building 127, Room 112
San Diego, CA 92132-5190

CONTRACT NO. N68711-98-D-5713
CTO NO. 0054

FINAL

ORDNANCE AND EXPLOSIVES WASTE/ GEOTECHNICAL CHARACTERIZATION REPORT

Revision 0

January 2, 2004

ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION, TIME-CRITICAL REMOVAL ACTION, AND GEOTECHNICAL AND SEISMIC EVALUATIONS AT INSTALLATION RESTORATION SITE 2 ALAMEDA POINT ALAMEDA, CALIFORNIA

DCN: FWSD-RAC-03-3604

Prepared by:

TETRA TECH FW, INC.
1230 Columbia Street, Suite 500
San Diego, CA 92101

REGISTERED PROFESSIONAL ENGINEER
ABID R. LOAN
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STATE OF CALIFORNIA

Abid Loan, P.E.
Project Manager

DEPARTMENT OF THE NAVY
SOUTHWEST DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
1220 PACIFIC HIGHWAY
SAN DIEGO, CA 92132-5190

5090
Ser 06CA.CD\0004
January 5, 2004

Ms. Anna-Marie Cook
US EPA
Region IX
75 Hawthorne Street
San Francisco, CA 94105-3901

Dear Ms. Cook:

This letter transmits the Final Ordnance and Explosives Waste/ Geotechnical Characterization Report for Installation Restoration Site 2, Alameda Point, Alameda, California.

Comments received for the Draft Final of this document were addressed and/or incorporated into this final document. Please insert and replace revised pages that are provided along with this transmittal letter that addresses those comments. Also, provided for your convenience is the final report cover and title page that replaces the draft final cover and title page. Please disregard draft final footnotes, as a new copy of the report will not be generated.

If you have any questions, please call Ms. Claudia Domingo, Remedial Project Manager at (619) 532-0935.

Sincerely,

THOMAS L. MACCHIARELLA
BRAC Environmental Coordinator
By direction of the Commander

Enclosures: (1) Cover and title page for the Final Ordnance and Explosives Waste/ Geotechnical Characterization Report for Installation Restoration Site 2, Alameda Point, Alameda, California.
(2) Revised document pages.

5090
Ser 06CA.CD\0004
January 5, 2004

Copy to:
Ms. Marcia Liao
Department of Toxic Substances Control
700 Heinz Avenue, Suite 200
Berkeley, CA 94710-2721

Ms. Judy Huang
San Francisco Bay Regional Water Quality Control Board
1515 Clay Street, Suite 1400
Oakland, CA 94612

Ms. Karla Brasaemle
EPA Consultant
Tech Law, Inc.
530 Howard Street, Suite 400
San Francisco, CA 94105

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ARRA Environmental Consultant
Northgate Environmental Management
950 Northgate Drive, Suite 313
San Rafael, CA 94903

5090
Ser 06CA.CD\0004
January 5, 2004

Blind copy to:
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Writer: C. Domingo, Code 06CA.CD, 2-0935
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TETRA TECH FW, INC.
1940 E. Deere Avenue, Suite 200
Santa Ana, CA 92705
Telephone 949-756-7500

TRANSMITTAL

To: Distribution

December 30, 2003

1990.054D.0419.02000

CTO 054

FINAL

ORDNANCE AND EXPLOSIVES WASTE/ GEOTECHNICAL CHARACTERIZATION REPORT
Revision 0
January 2, 2004

ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION, TIME CRITICAL REMOVAL ACTION, AND GEOTECHNICAL AND SEISMIC EVLAUATIONS AT INSTALLATION RESTORATION SITE 2
ALAMEDA POINT
ALAMEDA, CALIFORNIA

Please remove the pages indicated and replace with the enclosed pages. Please call 949-756-7541 with any questions. Put this sheet in the front of your route book to indicate that you have completed this update.

REMOVE PAGES	INSERT PAGES
Report Cover & Spine	Report Cover & Spine
Signature Page	Signature Page
Figure 1–2, following page 1-1	Figure 1–2, following page 1-1
Figure 2-1, IR Site Combined Topography/Bathymetry Map with Field Exploration Locations, following page 2-4	Figure 2-1, IR Site Combined Topography/Bathymetry Map with Field Exploration Locations, following page 2-4
Figure 2-2, IR Site Topographic Map, following page 2-5	Figure 2-2, IR Site Topographic Map, following page 2-5
Figure 3-1, IR Site 2 Search Grids, following page 3-1	Figure 3-1, IR Site 2 Search Grids, following page 3-1
Figure 3-2, IR Site 2 Exclusion Zone and Q-D ARC, following page 3-3	Figure 3-2, IR Site 2 Exclusion Zone and Q-D ARC, following page 3-3
Figure 3-5, OEW Locations on IR Site 2, following page 3-8	Figure 3-5, OEW Locations on IR Site 2, following page 3-8
Figure 4-7, Cross Section Location Map, following page 4-16	Figure 4-7, Cross Section Location Map, following page 4-16
Appendix M, Figure M-1	Appendix M, Figure M-1

TETRA TECH FW, INC.

TRANSMITTAL/DELIVERABLE RECEIPT

Contract No. N68711-98-D-5713 (RAC III)

Document Control No. 03-3604

File Code: 5.0

TO: Contracting Officer
Naval Facilities Engineering Command
Southwest Division
Ms. Beatrice Appling, 02R1.BA
1220 Pacific Highway
San Diego, CA 92132-5190

DATE: 01/06/04
CTO: 0040
LOCATION: Alameda Point

FROM: Edwin Neil Hart

Neil Hart, Program Manager

DESCRIPTION: Final Ordnance and Explosives Waste/Geotechnical Characterization Report, Rev. 0, 01/02/04

TYPE: ☐ Contract/Deliverable ☒ CTO Deliverable ☐ Notification ☐ Other

VERSION: Final (e.g. Draft, Draft Final, Final, etc.)

REVISION #: 0

ADMIN RECORD: (PM to Identify) Yes ☒ No ☐ Category ☐ Confidential ☐

SCHEDULED DELIVERY DATE: 10/17/03 ACTUAL DELIVERY DATE: 01/06/04

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Basic Contract File(02R1)		Date/Time Received
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TRANSMITTAL/DELIVERABLE RECEIPT

Contract No. N68711-98-D-5713 (RAC III)

Document Control No. 03-2899
File Code: 5.0

TO: Contracting Officer
Naval Facilities Engineering Command
Southwest Division
Ms. Beatrice Appling, 02R1.BA
1220 Pacific Highway
San Diego, CA 92132-5190

DATE: 10/30/03
CTO: 0054
LOCATION: Alameda Point

FROM: [signature]
Neil Hart, Program Manager

DESCRIPTION: Draft Final Ordance and Explosives Waste/Geotechnical Characterization Report, Rev. 0, 10/29/03

TYPE: ☐ Contract/Deliverable ☒ CTO Deliverable ☐ Notification
☐ Other

VERSION: Draft Final
(e.g. Draft, Draft Final, Final, etc.)

REVISION #: 0

ADMIN RECORD: Yes ☒ No ☐ Category ☐ Confidential ☐
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SCHEDULED DELIVERY DATE: 09/12/03 ACTUAL DELIVERY DATE: 10/30/03

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DEPARTMENT OF THE NAVY
SOUTHWEST DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
1220 PACIFIC HIGHWAY
SAN DIEGO, CA 92132-5190

5090
Ser 06CA.RW\1425
October 29, 2003

Ms. Anna-Marie Cook
US EPA
Region IX
75 Hawthorne Street
San Francisco, CA 94105-3901

Dear Ms. Cook:

Subj: SITE 2 OEW/GEOTECHNICAL CHARACTERIZATION REPORT

This letter transmits the Draft Final Ordnance and Explosives Waste/Geotechnical Characterization Report, Installation Restoration Site 2, for review by the Environmental Protection Agency. According to the Federal Facility Agreement, this draft final document has a 30-day review period. Please advise of any comments or concerns by November 12 , 2003, so Navy can release the final version of the report on November 29, 2003.

Thank you for your assistance.

Sincerely,

MICHAEL E. MCCLELLAND, P.E.
BRAC Environmental Coordinatior
By direction of the Commander

Encl: (1) *Draft Final Ordnance and Explosives Waste/Geotechnical Characterization Report, Ordnance and Explosive Wastes Characterization, and Geotechnical and Seismic Evaluations at Installation Restoration Site 2, Alameda Point, Alameda, California*

5090
Ser 06CA.RW\1425
October 29, 2003

Copy to ::
Ms. Marcia Liao
Department of Toxic Substances Control
700 Heinz Avenue, Suite 200
Berkeley, CA 94710-2721

Ms. Judy Huang
San Francisco Bay Regional Water Quality Control Board
1515 Clay Street, Suite 1400
Oakland, CA 94612

RESPONSE TO COMMENTS
DRAFT FINAL ORDNANCE AND EXPLOSIVES WASTE/ GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-03-2899, DATED DECEMBER 29, 2003, ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION, TIME-CRITICAL REMOVAL ACTION, AND GEOTECHNICAL AND SEISMIC EVALUATIONS AT INSTALLATION RESTORATION SITE 2, ALAMEDA POINT, ALAMEDA, CALIFORNIA

Comments by: Department of Toxic Substances Control (DTSC) 700 Heinz Avenue, Suite 200 Berkeley, CA 94710-2721	Responses by: Tetra Tech FW, Inc. 1940 E. Deere Avenue, Suite 200 Santa Ana, CA 92705
General Comments on Draft Final Ordnance and Explosives Waste/Geotechnical Characterization Report by DTSC	
Comment 1. Section 2, Wetland Assessment and Site Surveys: Please identify in map format, the locations of all the wetland areas such as Wetlands WE1, WE2 and WE3 (see Mr. Ram Ramanujan's original comment #2, dated April 2, 2003).	**Response 1.** Comment noted. As stated in the Draft Final Ordnance and Explosives Waste/Geotechnical Characterization Report, previously wetland areas designated as WE1, WE2, and WE3 are shown in Figure 1-2, IR Site 2 Site Plan. A reference to Figure 1-2 has been added in Section 2.1. The last sentence in the first paragraph of page 2-2 has been revised as follows: "Potential jurisdictional wetlands found within the project study area are listed in Table 2-1 and shown on Figure 1-2."
Comment 2. Please indicate that the true boundary of the landfill will be delineated and reported in the Remedial Investigation (RI) (We feel the true boundary of the landfill is a concern. But since a cap will be placed at Site 2, this becomes more of a RI issue).	**Response 2.** Comment noted. The Navy agrees that the landfill boundaries are an RI issue, and therefore, will be addressed in the RI report.
Comment 3. Please make sure the design document adequately address safety during any dirt moving. Specifically, please make sure that an approved Safety Submission is received from the Department of Defense Explosive Safety Board prior to dirt moving/excavation in the landfill area.	**Response 3.** Comment noted. Land use controls will be established during the CERCLA process, specifically, the development of the Proposed Plan and ROD. Appropriate engineering and institutional controls will address the landfill cap placement and construction and any excavation below the current land surface to mitigate potential risks associated with intrusive activities. An Explosives Safety Submission (ESS) was submitted by the Navy specifically for the time critical removal action activities related to surface clearance and excavation within the Possible OEW Burial Area. In the event of landfill cap placement and construction, and any excavation below the current land surface, the ESS can be amended to address additional risks and mitigation measures associated with intrusive activities during landfill cap construction.

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/ GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003, ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION, TIME-CRITICAL REMOVAL ACTION, AND GEOTECHNICAL AND SEISMIC EVALUATIONS AT INSTALLATION RESTORATION SITE 2, ALAMEDA POINT, ALAMEDA, CALIFORNIA

Comments by: Department of Toxic Substances Control (DTSC) 700 Heinz Avenue, Suite 200 Berkeley, CA 94710-2721	Responses by: Foster Wheeler Environmental Corporation 1940 E. Deere Avenue, Suite 200 Santa Ana, CA 92705
General Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by DTSC, Office of Military Facilities	
Comment 1. DTSC considers Site 2 a Solid Waste Management Unit (SWMU) subject to RCRA corrective action. Management of this unit must conform to RCRA, either directly or as ARARs. Please reflect this in the document.	**Response 1.** Comment noted. The scope of this report was to complete a surface ordnance and explosives waste (OEW) characterization of Installation Restoration (IR) Site 2 to locate, identify, and remove OEW on the ground surface of the site. This was required in order to safely perform geotechnical and seismic field evaluation tasks. Geotechnical and seismic evaluations were then conducted to characterize existing soil covers, identify seismic hazards, and perform preliminary engineering analyses. The OEW/Geotechnical Characterization Report specifically addresses the findings of the surface characterization and the geotechnical/seismic evaluations of IR Site 2. As stated in the *Final Focused Remedial Investigation Work Plan*, IR Site 2 [Foster Wheeler Environmental Corporation (FWENC), 2002], the scope of the geotechnical and seismic evaluations does not address chemical contamination in soil, sediment, or groundwater. There was no invasive work conducted as part of this existing scope to either characterize or delineate the area of refuse within the IR Site 2 disposal area. Removal of ordnance within the Possible OEW Burial Site excavation was followed by backfilling of the 2.5-acre excavation to original grade with clean soil and final compaction with a bulldozer, as stated in the *Final Focused Remedial Investigation Work Plan*, IR Site 2 (FWENC, 2002).

General Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by DTSC, Office of Military Facilities (Continued)

Comment 1. (cont.)	**Response 1.** (cont.)
	The heterogeneity of contaminant distribution and concentrations typically associated with landfills makes accurate characterization of landfill refuse impractical and virtually impossible. The Navy intends to provide containment at IR Site 2 in accordance with the presumptive remedy developed by the U.S. Environmental Protection Agency (EPA) as outlined in EPA Directive 9355.0-67FS. The Navy would also reiterate that the intent for closure at IR Site 2 is to follow the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) presumptive remedy for landfills as outlined in EPA Directive No. 5401F-93-035, which specifically states complete characterization of the landfill refuse is not required since containment is the preferred remedy. As previously stated, the Navy's intent for closure of IR Site 2 is to satisfy the seismic design requirements of Title 22 California Code of Regulations (CCR), which the Navy considers relevant and appropriate for closure of the landfill. A final determination of applicable or relevant and appropriate requirements (ARARs) will be addressed through the CERCLA remedy selection process. The ARARs will ultimately be set forth in the final Record of Decision for Operable Unit (OU)-4A, following issuance of a Proposed Plan and consideration of public comments received on the preferred remedial option for the site.

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/
GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003,
ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION,
TIME-CRITICAL REMOVAL ACTION,
AND GEOTECHNICAL AND SEISMIC EVALUATIONS
AT INSTALLATION RESTORATION SITE 2,
ALAMEDA POINT, ALAMEDA, CALIFORNIA

General Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by DTSC, (James Austreng, P.E.)

Comment	Response
Comment 1. The reports submitted to date indicate that no live OEW has been recovered within Site 2 or within the AIA. However, as indicated in previous memoranda (see attached, July 19, 2002), questions remain whether the boundaries of the area excavated and sifted were appropriately delineated.	**Response 1.** Comment noted. Following the discovery of 335 live 20mm high-explosives projectiles (and 14,304 inert rounds) during a radiological survey of IR Site 1 in 1998, the Navy ordered an Unexploded Ordnance (UXO) Site Investigation Survey, which was accomplished by Supervisor of Shipbuilding, Conversion and Repair, Portsmouth (SSPORTS) personnel in 1999. A surface search of IR Site 1 and IR Site 2 was completed during the initial phase of the survey to visually locate, identify, and remove all exposed OEW that could present a danger for subsequent survey phases. No ordnance and explosives (OE) or OEW were encountered during the surface search of the two sites. The grid networks established to complete the surface search/landfill delineations of the two sites are illustrated in the *Unexploded Ordnance Site Investigation Final Summary Report* (SSPORTS, 1999). After completing the surface search, UXO specialists used the MK 26 magnetometer to define the approximate boundaries of the landfill areas of both sites and Areas of Concern (AOC) within the landfill areas. The AOCs were determined from magnetometer readings, (large subsurface masses and discrete subsurface anomalies that could potentially be classified as UXO) historical data of waste disposal operations and interviews of personnel knowledgeable in the history of the sites. On IR Site 1, the AOC was the small arms range complex. On IR Site 2, the AOC (later referred to as the Possible OEW Burial Site) was established in the southeast portion of the site. The boundaries of the Possible OEW Burial Site were shown in the Final Summary Report (SSPORTS, 1999). This is the only location at IR Site 2 that was identified as a potential burial location for live ordnance. An area 20 to 25 percent larger than the Possible OEW Burial Site, as originally defined, was excavated and screened during the removal action. The excavated area completely encompassed the boundaries of the Possible OEW Burial Site. Because the live 20mm rounds found on IR Site 1 represented the only live ordnance ever found on both sites, they were designated the Most Probable Munition (MPM). Specific explosive characteristics of the MPM were used to determine Quantity-Distances (Q-D) and exclusion zones (EZ).

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/
GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003,
ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION,
TIME-CRITICAL REMOVAL ACTION,
AND GEOTECHNICAL AND SEISMIC EVALUATIONS
AT INSTALLATION RESTORATION SITE 2,
ALAMEDA POINT, ALAMEDA, CALIFORNIA

General Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by DTSC, (James Austreng, P.E.) (Continued)

Comment 1. (cont.)	**Response 1.** (cont.)
Furthermore, the efforts conducted as part of the OEW/Geotechnical characterization were restricted to a surface investigation. Consequently, uncertainty remains as to whether additional burial pits exist and/or whether live OEW may be located beneath land surface.	A comprehensive history of the amount and types of wastes deposited in the IR Site 2 landfill between 1952 and 1978 was published in the *Initial Assessment Study of Naval Air Station, Alameda, California, Final Report* by Ecology and the Environment for the Navy Assessment and Control of Installation Pollutants in 1983. Inert ordnance was reported to have been disposed of in the landfill by the Defense Logistics Agency, which left at least four loads of items of various categories and sizes in 1976. Additionally, the former Naval Air Station (NAS) Explosive Safety Manager, Mr. Winkleman, indicated that fired 20mm projectiles from the aircraft gun rework facility were disposed of in the IR Site 1 landfill areas while it was operating (SSPORTS, 1998) and that these disposal practices may have continued when the landfill at IR Site 2 became operational. The aircraft gun rework facility was a component of the larger Naval Aviation Depot (NAD). Repair and testing of aircraft guns took place on the second floor of Building 5 until the early 1980s, when it was moved to Building 29 on the southwest side of the lagoon (Delong, 2003). Inert target practice (TP) rounds were fired into large tanks of water as a part of the guns' operational tests. The expended TP rounds were regularly retrieved from the water tanks and taken to the landfill areas for disposal. The former NAS Explosive Safety Manager, Mr. Winkleman, indicated that fired 20mm projectiles were disposed of in the IR Site 1 landfill while it was operating (SSPORTS, 1998). The concentrated deposits of inert 20mm TP rounds found amid other rubble and debris during the removal action indicate that they were deposited in the landfill as a part of normal waste disposal activities, not in pits specifically excavated for the OEW emplacement. There is no historical record or any other indication of burial pits being used as a method of disposal for OEW in the landfill. There is no historical evidence that live OEW was ever placed in the IR Site 2 landfill, and none of the 8500+ 20mm rounds recovered during the removal action contained explosives or energetics.

RESPONSE TO COMMENTS DRAFT ORDNANCE AND EXPLOSIVES WASTE/ GEOTECHNICAL CHARACTERIZATION REPORT DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003, ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION, TIME-CRITICAL REMOVAL ACTION, AND GEOTECHNICAL AND SEISMIC EVALUATIONS AT INSTALLATION RESTORATION SITE 2, ALAMEDA POINT, ALAMEDA, CALIFORNIA	
General Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by DTSC, (James Austreng, P.E.) (Continued)	
Given such uncertainties, details of risk management measures must be incorporated into selection of preferred remedial action(s). These details should include not only the specific risk management/institutional measures to be taken, but also include information as to who will perform or be responsible to ensure the measures are implemented. In addition, a schedule for implementation of these measures as well as a reporting sequence should be outlined in the feasibility study.	Land use controls will be established during the CERCLA process, specifically, the development of the Proposed Plan and ROD. Appropriate engineering and institutional controls will address the landfill cap placement and construction and any excavation below the current land surface to mitigate potential risks associated with intrusive activities. Land use controls are discussed in Section 3.7 on page 3-8 of the document.
Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by DTSC, (James Austreng, P.E.)	
Comment 1. Section 1.5.2, Environmental Concerns and Mitigations, Page1-14. Text states "IR (Installation Restoration) Site 2 is currently used as a bird and wildlife sanctuary is proposed for transfer to the USFWS (United States Fish and Wildlife Service) for eventual use as a National Wildlife Refuge." It is not clear whether this proposed transfer will include the AIA. Should the AIA be excluded, additional investigation effort may be needed to determine whether buried OEW exist.	**Response 1.** Comment noted. None of the land that comprises the Additional Investigation Area (AIA) is within the original IR Site 2 landfill boundaries, and there is no historical record of the area ever being used for waste disposal operations. The AIA is predominately covered with roads and a runway apron and will be included in the transfer of IR Site 2 to the United States Fish and Wildlife Service (USFWS).

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/
GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003,
ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION,
TIME-CRITICAL REMOVAL ACTION,
AND GEOTECHNICAL AND SEISMIC EVALUATIONS
AT INSTALLATION RESTORATION SITE 2,
ALAMEDA POINT, ALAMEDA, CALIFORNIA

Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by DTSC, (James Austreng, P.E.) (Continued)

Comment 2. Page 1-18, Section 1.5.5.2, Design Basis While no live ordnance items were detected within the top one foot of soil within the boundaries of the landfill, the potential that other areas include live ordnance cannot be ruled out. Consequently, compaction efforts required for installation of the landfill cap must be taken into consideration the possibility that stressed imposed by heavy equipment may generate sufficient energy or movement that can trigger a detonation.	**Response 2.** Comment noted. There is no historical evidence of live OEW ever being buried in the West Beach Landfill during the time of its operation, none has ever been found there since the landfill closure, and none of the 8500+ 20mm rounds recovered during the removal action contained explosives or energetics. Disposal operations in the West Beach Landfill began in 1956 after the construction of the seawall. The landfill operated continuously until its closure in 1978, although unauthorized dumping continued until 1980. In 1985, the landfill cover was installed and compacted, and in 1986, 20,000 cubic yards of fill were placed atop the landfill and compacted. Later that year the landfill cover was graded to prevent ponding. The levee that surrounds the wetlands and the landfill was also constructed in 1986. The only record of waste disposed of in the southern wetlands pond is scrap metal. Some waste was deposited in the northern margins of the northern wetlands pond and was then covered with soil. The face of that buried waste defines the border between the landfill and the wetland. Based on the above information, risk of a detonation caused by compaction on the coastline, the levee, or the wetlands is minimal. The landfill area has been compacted and graded many times and was also used as a staging area for heavy equipment and materials for the construction of the levee, the riprap, and the culverts. The Possible OEW Burial Site, excavated during the removal action, was backfilled to original grade with clean soil and compacted with a bulldozer. Should future compaction efforts occur on the landfill area of IR Site 2, these would be undertaken after additional topsoil fill had been placed on the existing surface, which would subject buried items to less energy or movement than had already been applied during previous activities.

Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by DTSC, (James Austreng, P.E.) (Continued)

Comment	Response
Comment 3. Page 1-19, Section 1.5.6, Applicable Regulations and Criteria for OEW Management. The document failed to cite California Code of Regulations, Title 22 as a potential Applicable, or Relevant and Appropriate Requirement (ARAR). Conclusion: Based on the information provided, uncertainties remain regarding the potential presence of buried live OEW. Given such possibility, compaction efforts required for placement of the landfill cap must take into account the possibility that live ordnance may be present and could detonate due to stresses imposed by heavy equipment. Additionally, institutional controls and risk management measures must be included in the selection of the final remedial action.	**Response 3.** Comment noted. The *Final Focused Remedial Investigation Work Plan* (FWENC, 2002) for the project listed the following sections of CCR Title 22 as ARARs: Substantive requirements of 22 CCR, Section 66262.34 (pertaining to hazardous waste accumulation): • Hazardous waste generator requirements (22 CCR, Section 66262) • Container storage (22 CCR, Sections 66264.171 through 66264.178) • Transportation requirements (22 CCR, Section 66263) However, none of the OEW items recovered during the course of this project contained explosives, energetics, or other hazardous materials. The 20mm TP rounds were demilitarized by cutting them in half and they were disposed of in a Class III landfill. Because no hazardous waste of any type was generated, none of the criteria concerning the hazardous waste management requirements of CCR Title 22 were found applicable, or relevant and appropriate for the purposes of this document. A final determination of ARARs will be addressed through the CERCLA remedy selection process. The ARARs will ultimately be set forth in the final ROD for OU-4A following issuance of a Proposed Plan and consideration of public comments received on the preferred remedial option for the site. Institutional controls and risk management measures will also be included in the selection of the final remedial action.

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/
GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003,
ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION,
TIME-CRITICAL REMOVAL ACTION,
AND GEOTECHNICAL AND SEISMIC EVALUATIONS
AT INSTALLATION RESTORATION SITE 2,
ALAMEDA POINT, ALAMEDA, CALIFORNIA

Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by DTSC, (Ram Ramanujam, P.E.)

Comment 1. Section 1.5.5.1, State and Federal regulations. Installation Restoration (IR) Site 2 is classified as a hazardous waste landfill. The landfill closure systems should follow appropriate requirements of California Code of Regulations (CCR) Title 22. Section 1.5.5.1 should include reference to CCR Title 22.	**Response 1.** Comment noted. Title 22 CCR has not been determined to be applicable because the landfill has not been classified as hazardous (Class I). However, Title 22 CCR is still a relevant and appropriate requirement based on the nature of the wastes historically disposed of in the unit. Therefore, reference to Section 66264.25 (b) of Title 22 CCR pertaining to seismic design of hazardous waste landfills was added in Section 1.5.5.1.
Comment 2. Section 2, Wetland Assessment and Site Surveys. The Report should identify in map format, the locations of all the wetland areas such as Wetlands WE1, WE2, and WE3.	**Response 2.** Comment noted. Wetland areas designated as WE1, WE2, and WE3 are shown in Figure 1-2, IR Site 2 Site Plan. A reference to Figure 1-2 has been added in Section 2.1. The last sentence in the first paragraph of page 2-2 has been revised as follows: "Potential jurisdictional wetlands found within the project study area are listed in Table 2-1 and shown in Figure 1-2."
Comment 3. Table 4-6a, Summary of Material Design Parameters. The table provides Post-Earthquake/liquefaction Undrained Shear Strength values for various subsurface soil strata. It is not clear how these post-earthquake shear strength values were obtained from the laboratory tests. In this regard, please refer to the following publication: N. Ramanujam, L.L. Holish, and W.H. Chen, Post-earthquake Stability Analysis of Earth Dams (Earthquake Engineering and Soil Dynamics, Proceedings of the ASCE Geotechnical Engineering Division, Specialty Conference, June 19-21, 1978, Pasadena).	**Response 3.** Comment noted. Post-earthquake shear strength values were estimated from results of field and laboratory tests for this project and a literature search for properties of Young Bay Mud as discussed in Section 4.6.8 under subheading "Analysis Soil Profile and Parameters" (pages 4-37 and 4-38). The publication by Ramanujam et. al. was referenced in Section 4.6.8 (page 4-38).

Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by DTSC, (Ram Ramanujam, P.E.) (Continued)

Comment 4. Section 4.5.3, Page 4-19, 5th paragraph. "Maximum differential settlements were estimated by taking the difference between the settlement values calculated from the maximum assumed loading (landfill cap with additional fill) and the settlement caused by the minimum assumed loading (landfill cap only)." He definition of maximum differential settlement provided by the Report is incorrect. The Report evaluates settlement for two different conditions (landfill cap with and without additional fill). The difference between these two settlements will not yield differential settlement. The Report should be revised.	**Response 4.** Comment noted. The definition used for "maximum differential settlements" provides an upper bound estimate of the difference in settlement caused by the 4-foot-thick landfill cap and the 4-foot-thick landfill cap with an additional fill thickness of 10 feet in areas characterized by the different cone penetration test (CPT) hole locations. The estimated total settlements in areas that are relatively level will correspond to the settlement caused by either one of the loading conditions (landfill cap only or landfill cap with additional fill). However, it is possible that up to 10 feet of additional fill will be placed in an area causing differential settlements between adjacent areas (without the additional fill). This difference was tabulated in Table 4-6b for guidance. For further clarification, the term "differential settlement" will not be used and will be replaced with "difference in total settlements."
Comment 5. Tables 4-12a, 4-12b and 4-12c. It is not clear how the shear wave velocity values were assigned for various soil types used for the SHAKE91 computer analyses. This issue needs clarification.	**Response 5.** Comment noted. Tables 4-12a, 4-12b, and 4-12c show the shear-wave velocity values used for soil profiles 1, 2, and 3, respectively. As stated in Section 4.6.5.2, One-Dimensional Site Response Analyses, of the report: "Dynamic one-dimensional response analyses were performed for three 410-foot-thick "infinitely long" layered soil systems representing the site subsurface conditions at three CPT locations. These are: a) Profile 1 at CPT C-2-6 representing site soils along IR Site 2 southern boundary b) Profile 2 at CPT C-2-13 representing site soils along IR Site 2 western boundary c) Profile 3 at CPT C-2-19 representing site soils within the area between IR Sites 1 and 2"

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/
GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003,
ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION,
TIME-CRITICAL REMOVAL ACTION,
AND GEOTECHNICAL AND SEISMIC EVALUATIONS
AT INSTALLATION RESTORATION SITE 2,
ALAMEDA POINT, ALAMEDA, CALIFORNIA

Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by DTSC, (Ram Ramanujam, P.E.) (Continued)

Comment 5. (cont.)	**Response 5.** (cont.)
	The shear-wave velocity values were assigned by taking an average value (for different depth intervals) along velocity profiles obtained from either field measurements using seismic CPT for this project or from the available data for nearby projects. The selected depth intervals consisted of one or more layers in the discretized soil column used in SHAKE91 analyses, depending on the variability of velocity profiles. The shear-wave velocities for soils less than 100 feet deep were derived from direct measurements for this project. Shear-wave velocities for soils more than 100 feet deep were assigned from reported values from the San Francisco-Oakland Bay Bridge (SFOBB) project. The following text from Section 4.6.5.2 (page 4-30) describes the process in more detail: "The unit weight and shear-wave velocity profiles used in the dynamic site response analyses, summarized in Tables 4-12a, b, and c, were derived from the site-specific field and laboratory test results obtained for IR Site 2 and the area between IR Site 1 and 2 soils during this investigation (generally at depths less than 100 feet), and the data provided for the SFOBB project for the deeper soil layers to the depth of bedrock (Fugro-EMI, 2001a; 2001b). Field exploration including CPT soundings and soil borings were performed at the site to measure in situ penetration resistance and seismic-wave velocities and to recover soil samples for measuring in situ moisture and density. The unit weight and shear-wave velocity of the foundation Franciscan Formation bedrock were assumed to be to 140 pcf and 5,000 feet/sec^2, respectively." Shear-wave velocity measurements at CPT locations, C-2-6, C-2-13, and C-2-19, can be found in Appendix B. The CPT location ID #, noted on the logs, are CPT-06seis, CPT-13seis, and CPT-753, which correspond to sample location ID # C-2-6, C-2-13, and C-2-19, respectively (these cross references are noted on Table 4-2, Survey Coordinates of Sample Locations).

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/
GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003,
ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION,
TIME-CRITICAL REMOVAL ACTION,
AND GEOTECHNICAL AND SEISMIC EVALUATIONS
AT INSTALLATION RESTORATION SITE 2,
ALAMEDA POINT, ALAMEDA, CALIFORNIA

Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by DTSC, (Ram Ramanujam, P.E.) (Continued)

Comment	Response
Comment 6. Figures 4-4 and 4-5. These figures should include the elevation of the water table, and Subsurface cross section profiles should include Standard Penetrometer Test (SPT) results.	**Response 6.** Comment noted. The elevation of the water table will be added to Figures 4-4 and 4-5. Interpreted subsurface soil profiles (Figures 4-8a to 4-8i) already include detailed information on subsurface conditions and standard penetration test (SPT) results for borings drilled at the site. Inclusion of SPT results in Figures 4-4 and 4-5 (Geologic Cross Sections) was not considered helpful since this data can be presented more clearly in Figures 4-8a to 4-8i, and because its inclusion will overcrowd Figures 4-4 and 4-5.
Comment 7. Appendix L: One-Dimensional Site Response and Liquefaction-Induced Deformation Analyses. The Report uses the empirical method developed by Bartlett and Youd, 1995 and Youd et.al., 2002 to estimate the magnitude of lateral spread displacements for the potentially liquefied soils. However, the empiracle method is applicable only for "free face" slope conditions. The assumed "free face" is partially covered by the bay water and it cannot be considered a "free-face". The Report should revisit the deformation analyses.	**Response 7.** Comment noted. The definition of the "free face" slope condition in the literature on the topic of liquefaction and lateral spreading includes embankments, quay walls (constructed at ports), stream or river banks, embankment slopes of canals, and so forth (Bardet et al., 2002; Rauch, 1997). Once liquefaction transforms a subsurface liquefiable layer into a fluidized mass, gravity plus inertial forces that result from the earthquake may cause the mass to move down-slope toward a cut slope or free face (such as a river channel or a canal). Lateral spreads most commonly occur on gentle slopes that range between 0.3 and 5 percent, and commonly displace the surface by several meters to tens of meters. The geologic conditions conducive to lateral spreading (gentle surface slope, shallow water table, liquefiable cohesionless soils) are frequently found along streams and other waterfronts in recent alluvial or deltaic deposits, as well as in loosely placed, saturated, sandy fills (Youd and Hoose, 1976). Surface displacements proceed down-slope or toward a steep free face (such as a stream bank) with the formation of fissures, scarps, and grabens (Rauch, 1997). Figure 3.1 of the Alan F. Rauch's PhD thesis (Rauch, 1997) clearly shows that a "free face" condition includes river or stream banks or any waterfront slope (see Attachment 1 at the end of these Response to Comments.)

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/
GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003,
ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION,
TIME-CRITICAL REMOVAL ACTION,
AND GEOTECHNICAL AND SEISMIC EVALUATIONS
AT INSTALLATION RESTORATION SITE 2,
ALAMEDA POINT, ALAMEDA, CALIFORNIA

REFERENCES APPLICABLE TO RESPONSE TO COMMENTS

Bardet, J. P., T. Tobita, N. Mace, and J. Hu. 2002. Regional Modeling of Liquefaction-Induced Ground Deformation. Earthquake Engineering Research Institute. *Earthquake Spectra*. Vol. 18. No. 1. February. pp. 19-46.

Delong, Doug. 2003. Interview conducted 16 June 2003 concerning the past use/existence of the gun rework facility on NAS Alameda.

Ecology and Environment. 1983. *Initial Assessment Study of Naval Air Station, Alameda, California, Final Report*. Prepared for Navy Assessment and Control of Installation Pollutants and Naval Energy and Environmental Support Activity, Port Hueneme, California.

Foster Wheeler Environmental Corporation (FWENC). 2002. *Final Focused Remedial Investigation Work Plan*. Ordnance and Explosives Waste Characterization, and Geotechnical and Seismic Evaluations at Installation Restoration Site 2. Alameda Point, Alameda, California.

Fugro-EMI. 2001a. Ground Motion Report, SFOBB East Span Seismic Safety Project. Unpublished Report for California Department of Transportation: Sacramento, California. February.

Fugro-EMI. 2001b. Final Yerba Buena Island Geotechnical Site Characterization Report, San Francisco-Oakland Bay Bridge, East Span Seismic Safety Project. Unpublished Report for California Department of Transportation: Sacramento, California. December.

Rauch, A. F. 1997. EPOLLS: An Empirical Method for Predicting Surface Displacements Due to Liquefaction-Induced Lateral Spreading in Earthquakes. PhD Dissertation, Virginia Tech, Civil Engineering Department.

	Supervisor of Shipbuilding, Conversion and Repair, Portsmouth (SSPORTS). 1998. *Unexploded Ordnance Emergency Removal Action, Installation Restoration Site 1, Alameda Point, Alameda, California, Summary Report*. Vallejo, California. SSPORTS. 1999. *Unexploded Ordnance Site Investigation Final Summary Report, Final*. Vallejo, California. Youd, T. L. and S. N. Hoose. 1976. Liquefaction During 1906 San Francisco Earthquake. *Journal of Geotechnical Engineering Division*. ASCE, Vol. 102. No. 102. No. GT5. May. pp. 425-439.

ATTACHMENT 1

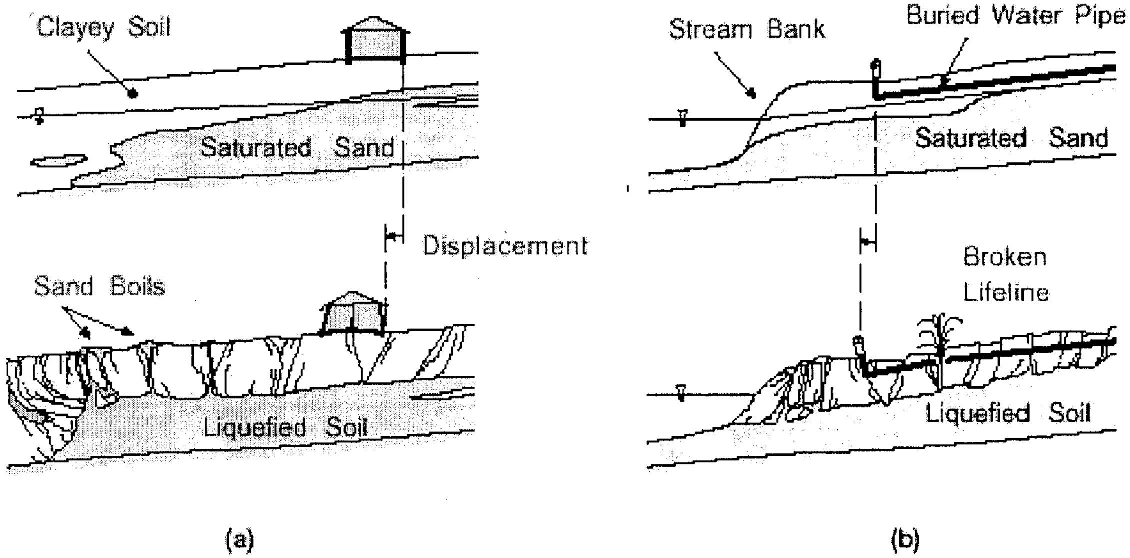

Figure 3.1. Soil liquefaction and lateral spreading of (a) gently sloping ground and (b) toward a free face.

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/
GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003,
ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION,
TIME-CRITICAL REMOVAL ACTION,
AND GEOTECHNICAL AND SEISMIC EVALUATIONS
AT INSTALLATION RESTORATION SITE 2,
ALAMEDA POINT, ALAMEDA, CALIFORNIA

Comments by: Environmental Protection Agency (EPA) 75 Hawthorne Street San Francisco, CA 94105-3901	Responses by: Foster Wheeler Environmental Corporation 1940 E. Deere Avenue, Suite 200 Santa Ana, CA 92705
General Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by EPA	
Comment 1. It appears that some of the cone penetrometer test (CPT) results may not be included in the Draft Ordnance and Explosives Waste Characterization, Time-Critical Action, and Geotechnical and Seismic Evaluations at Installation Restoration Site 2 Report (the report). Specifically, the logs for C-2-16 through C-2-21 appear to be missing. There are a second set of logs numbered 750 through 753, 757 and 758 in Appendix B, but these labels do not correspond to the CPT locations in Table 4-1 or on Figure 2-1, so it is not clear that these are the missing logs.	**Response 1.** Comment noted. CPT logs for C-2-16 through C-2-21 correspond to logs numbered CPT-750 through CPT-753, CPT-757, and CPT-758 in Appendix B. The cross-references are listed in Table 4-2 (Survey Coordinates of Sample Locations). Table 4-2 lists the "Sample Location ID #" (designation used in the text and figures) and its corresponding "CPT location ID #" (as recorded in the CPT logs). For added clarity, the "Survey Point Number" (used by the surveyors) and "CPT location ID #" (as recorded in the CPT logs) for each CPT hole location have been added to Table 4-1. Also, the following text will be added to Section 4.1.1 (Cone Penetration Testing): "The sampling location designations used in the report may not coincide with the designation used by the surveyors or those reported in the logs. Therefore, cross-references between the "Sample Location ID #" (designation used in the text and figures) and its corresponding "Survey Point Number" (used by the surveyors) and "CPT location ID #" (as recorded in the CPT logs) are included in Table 4-1."

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/
GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003,
ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION,
TIME-CRITICAL REMOVAL ACTION,
AND GEOTECHNICAL AND SEISMIC EVALUATIONS
AT INSTALLATION RESTORATION SITE 2,
ALAMEDA POINT, ALAMEDA, CALIFORNIA

General Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by EPA (Continued)

Comment	Response
Comment 1. (cont.) The results of CPT probe C-2-16, which do not appear to be included in Appendix B, are of particular interest as this probe was advanced directly adjacent to Boring B-2-13. Please revise the report to include all of the applicable CPT logs. Also, please specify where the locations for CPT logs 750 through 753, 757 and 758 are located. In addition, please discuss the accuracy of the correlation used to interpret the CPT logs by discussing the correlation between the log of CPT-2-16 and the boring log for boring B-2-13.	**Response 1.** (cont.) The CPT and boring performed at Location C-2-16 (identified as CPT-757 on the CPT log) and B-2-13 showed consistent results. The initial 12 feet below ground surface (bgs) consisted of very hard fill material (riprap or other debris). A pre-boring had to be performed before the CPT could begin, and no samples were obtained due to the hard material encountered. From 12 to 25 feet bgs, the recorded tip resistance was in excess of 100 tons per square foot (tsf), and a soil classification of sand was recorded on the CPT logs. These are consistent with the recorded soil boring blow counts which ranged from N = 7 to 44. Between 25 to 55 feet bgs, very low tip resistance (less than 10 tsf) was recorded in the CPT log indicating predominantly fine-grained material (silts and clay) in this region. The blow counts at this depth were also low (N = 2 to 8) and high-plasticity clay was observed in the samples taken. Between 55 and 85 feet bgs, the soil was classified (in the CPT logs) as predominantly sand with pockets of silty sand. High tip resistance (greater than 250 tsf) was recorded at around 60 feet bgs which indicates very stiff materials (sands and gravel). This finding was confirmed by the borings where there were difficulties in obtaining samples at 60 feet bgs. The pockets of silty sand were confirmed with lower blow counts (N = 5 to 14) at 80 feet bgs and the silty sand sample that was observed. From 85 to 185 feet bgs, the CPT logs recorded consistently low tip resistance readings (20-50 tsf), indicating the presence of predominantly fine-grained material (silts and clays). This was confirmed by borings performed at 90 and 100 feet bgs, where clay samples were recovered and an "easy push" was observed when shelby tubes were extracted. Based on the similar findings between the CPT and borings performed, the accuracy of the correlation used to interpret the CPT logs was considered adequate. The presence of fill material mixed in with waste was not recorded in the CPT logs because the classification was based on a correlation only with probable soil behavior types (Robertson and Campanella, 1985).

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/
GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003,
ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION,
TIME-CRITICAL REMOVAL ACTION,
AND GEOTECHNICAL AND SEISMIC EVALUATIONS
AT INSTALLATION RESTORATION SITE 2,
ALAMEDA POINT, ALAMEDA, CALIFORNIA

Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by EPA

Comment 1. *Note:* There is no Specific Comment 1.	**Response 1.** No response needed.
Comment 2. Section 1.1.2, Site History, Page 1-2. This section notes that (Ordnance and Explosives Waste) "OEW may have also been deposited in the 2.5-acre (approximate) Possible OEW Burial Site located in the southern part of the landfill." The use of the word "also" would seem to indicate that it is suspected that OEW was deposited in the remainder of the landfill in the past. It has already been noted in this section that "inert ordnance" was placed in the landfill. Experience has shown that the deposition of "inert ordnance" in a landfill almost inevitably results in the intentional/unintentional inclusion of live ordnance as well.	**Response 2.** Comment noted. The word "also" was used to indicate that both inert ordnance and OEW may have been buried in the West Beach Landfill. When the document was written, the definition of inert ordnance did not include OEW. A comprehensive history of the amount and types of wastes deposited in the IR Site 2 landfill between 1952 and 1978 was published in the Initial Assessment Study of Naval Air Station, Alameda, California, Final Report by Ecology and the Environment, for the Navy Assessment and Control of Installation Pollutants in 1983. Inert ordnance were reported to have been disposed of in the landfill by the Defense Logistics Agency, who left at least four loads of items of various categories and sizes in 1976. Inert ordnance [as defined in Naval Sea Systems Command (NAVSEA) OP 5] is "(a) ammunition and components with all explosive material removed and replaced with inert material, (b) empty ammunition or components, or (c) ammunition or components that were manufactured with inert material in place of all explosive material" (NAVSEA, 2001). Inert items are often manufactured for classroom training aids, display cases, and other educational and training purposes. At the time the document was written, the acronym OEW defined materials that are now attributed to the acronym "MC" (Munitions Constituents), which can be found in 10 United States Code (USC), 2710 (e)(4). The citation defines MC (OEW in the document) as "any materials originating from unexploded ordnance, discarded military munitions, or other military munitions, including explosive and non-explosive materials, and emission, degradation, or breakdown elements of such ordnance or munitions" [10 USC 2710 (e)(4)]. Examples of OEW include shell casings, powder containers, or expended rocket motors. Expended 20mm shell casings and target practice (TP) rounds from the aircraft gun rework facility may have been buried in the IR Site 2 landfill. The former NAS Explosives Safety Manager, Mr. Winkleman, indicated that fired 20mm projectiles were disposed of in the IR Site 1 landfill while it was in operation. [Supervisor of Shipbuilding, Conversion and Repair, Portsmouth (SSPORTS), 1998].

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/
GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003,
ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION,
TIME-CRITICAL REMOVAL ACTION,
AND GEOTECHNICAL AND SEISMIC EVALUATIONS
AT INSTALLATION RESTORATION SITE 2,
ALAMEDA POINT, ALAMEDA, CALIFORNIA

Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by EPA (Continued)	
Comment 2. (cont.)	**Response 2.** (cont.)
	There is no historical evidence, however, that live ordnance or explosives were ever placed in the IR Site 2 landfill, and none of the 8500+ 20mm rounds recovered during the removal action contained explosives or energetics.
The Unexploded Ordnance (UXO) Site Investigation Final Summary Report Operable Unit #3 (IR Sites 1 and 2) at Alameda Point, dated October 22, 1999, indicated that 335 20-mm high explosive (HE) projectiles were recovered during a removal action conducted at Site IR-1 in 1998. The presence of UXO (containing HE) at a site where records indicate that only inert ordnance was disposed of is a further indicator of the potential for other UXO or live ordnance and explosives (OE) to be located elsewhere on both Sites IR-1 and IR-2. This potential presence of UXO/OE at Site 2 should be noted in the history to ensure that all concerned are aware of this and that it is considered in any intrusive activities accomplished in the area in the future.	On IR Site 1, the 335 live 20mm HE rounds discovered during the course of a radiation survey were located on the ground surface or in very shallow excavations immediately adjacent to the rounds on the surface. All were found in the immediate area of the pistol range impact berm. The fact that the pistol range was constructed over a former disposal area of the IR Site 1 landfill after soil cover was placed atop it would seem to indicate that the live rounds were not landfill remnants, but were Discarded Military Munitions (DMM) placed there after that portion of the landfill was no longer in use. A surface characterization of the uplands found no live OE/OEW, and none were found during the time-critical removal action. Information regarding the discovery of abandoned UXO/OE on IR Site 1 and the potential for other similar abandoned ordnance to exist on Site 2 or elsewhere on NAS Alameda will be addressed in Section 1.1.2 of the document.
Please expand this section to include information identifying the potential presence of UXO/OE in the entire IR Site 2.	Vegetation at the West Beach Landfill (IR Site 2) was removed or cut to a height of 4 inches to facilitate conducting a 100-percent surface search of the upland areas prior to the start of removal action activities. Aside from an inert training land mine found near the wetlands boundary, no OEW items were located. A discussion regarding the discovery of live ordnance on IR Site 1 and the potential for abandoned ordnance to be found at IR Site 2 will be included in Section 1.1.2.

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/
GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003,
ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION,
TIME-CRITICAL REMOVAL ACTION,
AND GEOTECHNICAL AND SEISMIC EVALUATIONS
AT INSTALLATION RESTORATION SITE 2,
ALAMEDA POINT, ALAMEDA, CALIFORNIA

Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by EPA (Continued)

Comment 3. Section 1.4, Data Quality Objectives Process, Subsection OEW, Page 1-11. This section provides a limited overview of the process for establishing search grids and conducting a surface sweep, but does not give any of the details as to exactly how these actions were accomplished, or where this information may be found. While these details are later provided in Section 3.0, Ordnance and Explosives Waste Characterization, this is not apparent unless the reader has turned to the Table of Contents in search of this information. Please include a statement in this section identifying where the details of how the search grids were established and how the surface sweep was conducted may be found.	**Response 3.** Comment noted. A reference to Section 3.0 of the document has been added in Section 1.4, Data Quality Objectives Process, Subsection OEW, Page 1-11.
Comment 4. Table 1-2, Data Quality Objectives for Ordnance and Explosives Concerns. In the section of this table entitled "Step 1, Statement of the Problem," the statement is made that "OEW/UXO may have been buried in the landfill portion of IR Site 2." This followed by a statement that "No OEW is expected to be encountered." These two statements appear to directly contradict each other. Please review the two statements and correct them, or provide an explanation as to why they are not contradictory.	**Response 4.** Comment noted. In Table 1-2, The first statement in the 'Step 1' column that refers to OEW will be revised to read: "*Spent* OEW/UXO may have been buried in the landfill portion of IR Site 2." The last statement in the column will also be revised to read: "No *live* OEW is expected to be encountered."

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/
GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003,
ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION,
TIME-CRITICAL REMOVAL ACTION,
AND GEOTECHNICAL AND SEISMIC EVALUATIONS
AT INSTALLATION RESTORATION SITE 2,
ALAMEDA POINT, ALAMEDA, CALIFORNIA

Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by EPA (Continued)

Comment 5. Table 1-3, Data Quality Objectives for Geotechnical Concerns. The Statement of the Problem indicates that, "No contamination of soil or groundwater exceeds the TTLC hazardous levels." However, the description of the wastes disposed of in the landfill indicate that they included paint, paint sludges, batteries, drums of unknown chemicals, polychlorinated biphenyl (PCB) contaminated liquids, radium dials, tear gas agents and surplus pesticides. While the Navy has not characterized any of this waste, it should consider that characteristically toxic, ignitable, corrosive and reactive, as well as listed, hazardous wastes are likely present within the landfill and should close the landfill accordingly. Please revise the statement of problem column to further clarify the significance of the absence of any characterization data on the waste contained within the landfill and state explicitly that an uncontrolled release of these wastes to San Francisco Bay would be unacceptable.	**Response 5.** Comment noted. The Statement of Problem (see Table 1-3) regarding contamination of soil and groundwater has been revised to read: "Waste depth is unknown. Waste delineation is not part of the geotechnical characterization. Contamination of soil or groundwater exceeding the TTLC hazardous levels is not anticipated." A description of the waste disposed in the landfill was included in Section 1.1.2 (Site History). The intent for closure at IR Site 2 is to follow the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) presumptive remedy for landfills as outlined in EPA Directive No. 5401F-93-035, which specifically states that complete characterization of the landfill refuse is not required since containment is the preferred remedy. The potential for waste release into the San Francisco Bay is a concern; therefore, it was identified as the Remedial Action Objective for the Geotechnical FS Report [Foster Wheeler Environmental Corporation (FWENC), 2002]. The Statement of Problem (see Table 1-3), regarding seismic and geotechnical evaluation, has been revised to read: "Seismic and geotechnical evaluation is needed to determine the potential for slope failure into San Francisco Bay. Slope failure is a concern due to the potential release of waste into the bay."

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/
GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003,
ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION,
TIME-CRITICAL REMOVAL ACTION,
AND GEOTECHNICAL AND SEISMIC EVALUATIONS
AT INSTALLATION RESTORATION SITE 2,
ALAMEDA POINT, ALAMEDA, CALIFORNIA

Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by EPA (Continued)

Comment 6. Table 3-1, Maximum Case Fragment Ranges for Selected Single Item Detonations; and Table 3-2, Inhabited Buildings and Public Traffic Route Distances. While the data in the subject tables are correct, the source cited is "Naval Sea Systems Command, 1995." It is assumed the referenced document should be Naval Sea Systems Command (NAVSEA) OP 5, Ammunition and Explosives Ashore, Safety Regulations for Handling, Storing, Production, Renovation and Shipping, Revision 7, 15 January 2001. These two tables appear to be extracts of Tables 13-2 and 7-7 in revision 7 of the OP 5. The cited reference (same document, Revision 6, 1 March 1995) was superceded by the 15 January 2001 Revision 7. Please correct this reference in the two tables and in Section 6.0, References, page 6-5.	**Response 6.** Comment noted. Revision 7 was used as a reference for the creation of the document, but the correct publication date was not reflected in the citations or the reference. The two citations and the reference will be corrected.

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/
GEOTECHNICAL CHARACTERIZATION REPORT
DCN: FWSD-RAC-02-1787, DATED JANUARY 20, 2003,
ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION,
TIME-CRITICAL REMOVAL ACTION,
AND GEOTECHNICAL AND SEISMIC EVALUATIONS
AT INSTALLATION RESTORATION SITE 2,
ALAMEDA POINT, ALAMEDA, CALIFORNIA

Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by EPA (Continued)

Comment 7. Section 3.5, QC Procedures, Page 3-5. In the sixty paragraph of this section, it is unclear as to the frequency/percentage of the grids selected for the Search Effectiveness Probability (SEP) Tests. As a result, it is unknown as to the intensity of the contractor's QC program. In addition, no mention is made of the Quality Assurance (QA) process established by the Navy to ensure the data reported by the contractor was within established parameters. Please expand this section to include the details of the contractor's QC program or identify where these details may be found. In addition, please provide the details of the Navy QA program or identify where it may be found.	**Response 7.** Comment noted. The UXO Characterization Teams received certification to conduct surface characterization operations on IR Site 2 upon successful completion of the Search and Effectiveness Probability (SEP) test. The teams were required to demonstrate the capability to achieve an 85 percent probability of detection with a 90 percent confidence level. Subsequent to their initial certification, periodic SEP tests were conducted to monitor the continued effectiveness of each team, initially at a frequency of two tests per month. A SEP test was also administered when the search team personnel composition changed. The results of every SEP (pass/fail) test were documented in the Contractor Quality Control Reports (CQCRs), which were submitted daily to the Navy Resident Officer In Charge of Construction (ROICC), the Remediation Project Manager (RPM), and the Environmental Compliance Manager (ECM). The project QC team was comprised of the UXO QC Representative (USACE quality assurance (QA)/quality control (QC)-certified), the Senior UXO Supervisor, the Project Quality Control Manager (PQCM), and the Project Manager. All were responsible for implementing QC procedures contained in the Contractor Quality Control (CQC) Plan, which was an appendix of the *Final Focused Remedial Work Plan* (Work Plan). All distinguishable aspects of the project that required measures to verify the quality of work performed and compliance with specified requirements were identified as definable features of work (DFWs), and controls for each DFW were assigned. The CQC Plan implemented preparatory, initial, and follow-up control phases for all aspects of every DFW.

RESPONSE TO COMMENTS
DRAFT ORDNANCE AND EXPLOSIVES WASTE/ GEOTECHNICAL CHARACTERIZATION REPORT
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Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by EPA (Continued)

Comment 7. (cont.)	**Response 7.** (cont.)
	The NAVFAC Southwest Division Naval Facilities Engineering Command QA Officer reviewed the CQC Plan to ensure it was in compliance with the requirements of Naval Facilities (NAVFAC) P-445 [Construction Quality Management (CQM) Program] (NAVFAC, 2000), Unified Facilities Guide Specification (UFGS)-D 01450H (NAVFAC, 2003). Changes were made to the latest revision of NAVFAC P-445 to bring it and UFGS-D 01450H into agreement. The QA Officer was required to approve the CQC Plan prior to its implementation. SWDIV recommendations for improvements to the CQC Plan were incorporated into the draft version of the plan, and it was further refined during the review process. Additional Navy oversight of the QC process was provided by the Naval Ordnance Safety and Security Activity (NOSSA) who reviewed the PCQC Plan, the Work Plan, and the Action Memorandum. Their comments and recommendations were incorporated into the documents. The information regarding the SEP test frequency, the contractor's, and the Navy's QA/QC programs will be included in the report. As a part of SWDIV QA oversight, the ROICC was notified prior to the administration of every SEP test so that the ROICC or a staff member could observe the test-grid preparation and conduct of the evaluation. Additionally, the SEP tests and other portions of the CQC Plan that affected other aspects of ongoing site activities were discussed during weekly CQC meetings between the ROICC, RPM, ECM, and the Contractor. These meetings were held to further ameliorate the QA/QC process by identifying elements of the plan that could be modified to optimize the realized results.

RESPONSE TO COMMENTS
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Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by EPA (Continued)	
Comment 8. Table 4-6a. Summary of Material Design Perimeters. The 1949 United States Geological Survey (USGS) quadrangle map indicates that the entire Installation Restoration Site 2 was under water. Therefore, the minimum elevation range of the fill strata must be 0 to +20 feet above mean sea level (msl), though it is likely to range down to 10 to 20 feet below msl. Please revise the table to indicate that the elevation range of the fill extends down to at least 0 feet msl. Please attempt to find historical navigation charges of the area to determine how much fill was actually placed at the site.	**Response 8.** Comment noted. Comment noted. Table 4-6a will be revised to reflect the correct ranges of elevation and thickness values for the fill, Young Bay Mud, and Merritt Sand layers based on the subsurface profiles presented in Figures 4-4, 4-5, and 4-8a through 4-8i that provide correct elevation and thickness values.

Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by EPA (Continued)

Comment	Response
Comment 9. Section 4.6.5, Ground Response Analyses, Page 4-26. It is unclear why a site design maximum rock acceleration of 0.04g was chosen. Based on the earthquake attenuation relationships reported in Section 4.6.5, the maximum expected bedrock acceleration at the site is 0.36g. The report also indicates that, "However since the historical seismicity data indicates that the site has experienced a maximum rock acceleration of about 0.4g in the past 200 years, (due to the 1906 San Francisco earthquake on the San Andreas Fault, see Section 4.6.1), a site design maximum rock acceleration of 0.40g was assumed for seismic evaluations." However, this 0.40g acceleration is not actually historical data, rather it is based on an attenuation relationship reported in Section 4.6.1, but not in Section 4.6.5. This raises the question of why the attenuation relationship (Bozorgnia/Campbell/Niazi) used in Section 4.6.1 to evaluate earthquakes on the San Andreas fault was not also applied to earthquakes on the Hayward fault. Please revise the report to provide further clarification of the applicability of the Borzorgnia/Campbell/Niazi attenuation relationship to the site and why it was not used to evaluate site impacts from an earthquake originating at the Hayward fault.	**Response 9.** Comment noted. As the reviewer has noted, the estimated rock acceleration of 0.4g at the project site due to the 1906 San Francisco earthquake on the San Andreas Fault is not a recorded historical acceleration. However, it is common practice in the industry to use reasonable estimates of the site historical rock acceleration based on the estimated earthquake magnitude, site epicentral distance, and recent attenuation relations. As suggested by the reviewer, additional deterministic calculations using the 1999 Bozorgnia/Campbell/Niazi attenuation equation were performed, and these results will be added to Section 4.6.5 of the report. The attenuation equation was used for both San Andreas and Hayward faults and the results will be included in Table 4-10 of the report. Based on these new calculations, the 1999 Bozorgnia/Campbell/Niazi attenuation equation for hard rock results in a median peak horizontal ground acceleration (PHGA) of approximately 0.37g at the site for a magnitude 7.9 earthquake on the San Andreas Fault. Due to uncertainties associated with any seismic hazard analysis, the common practice in industry is to round the estimated peak ground acceleration to only one decimal point and usually to the nearest tenth higher than the estimated acceleration. The computed PHGAs for the site are median values, and the median plus one standard deviation were calculated to be approximately up to 0.2g higher than the estimated median value (approximately 0.5g to 0.6g). Therefore, the selected site design peak rock acceleration of 0.4g due to a magnitude earthquake of 7.9 on the San Andreas Fault, which is slightly higher than the estimated median acceleration value, is considered a reasonable design value for this site and the level of risk involved.

RESPONSE TO COMMENTS
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ALAMEDA POINT, ALAMEDA, CALIFORNIA

Specific Comments on Draft Ordnance and Explosives Waste/Geotechnical Characterization Report by EPA (Continued)	
Comment 9. (cont.)	**Response 9.** (cont.) Table 4-10 shows that the estimated site accelerations due to Hayward and San Andreas faults at a distance of approximately 11.2 kilometers (km) and 18.7 km from the site, respectively, are approximately equal. Therefore, for estimated peak rock accelerations in Table 4-10, liquefaction and slope instability hazards at the site are more influenced by the selected magnitude 7.9 design earthquake on the San Andreas Fault, rather than the magnitude 7.1 earthquake on the closer Hayward Fault. The larger magnitude earthquake at a farther distance from the site results in a longer duration of shaking and thus more severe liquefaction and slope instability hazards.
ERRATA	
Comment 1. Table 4-14, Summary of Slope Stability Analysis Results Please do not report seismic slope stability factors of safety in a column headed "Static Factor of Safety."	**Response 1.** Comment noted. The column heading has been revised to "Static and Pseudo-Static Factor of Safety."

REFERENCES APPLICABLE TO RESPONSE TO COMMENTS

Ecology and Environment. 1983. *Initial Assessment Study of Naval Air Station, Alameda, California, Final Report.* Prepared for Navy Assessment and Control of Installation Pollutants and Naval Energy and Environmental Support Activity, Port Hueneme, California.

Foster Wheeler Environmental Corporation (FWENC). 2002. *Preliminary Draft Geotechnical Feasibility Study Report.* Geotechnical and Seismic Feasibility Study Report for Installation Restoration Site 2. Alameda Point, Alameda, California.

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Naval Sea Systems Command (NAVSEA). 2001. *Ammunition and Explosives Ashore Safe Regulations for Handling, Storing, Production, Renovation, and Shipping.* U.S. Navy Manual (NAVSEA) OP-5. Revision 7. January.

Robertson, P. K. and R. G. Campanella. 1985. Liquefaction Potential of Sands using the Cone Penetration Test. *Journal of the Geotechnical Engineering Division.* ASCE, Vol. 111, No. 3, March. pp. 384-403.

Supervisor of Shipbuilding, Conversion and Repair, Portsmouth (SSPORTS). 1998. *Unexploded Ordnance Emergency Removal Action, Installation Restoration Site 1, Alameda Point, Alameda, California Summary Report.* Vallejo, California.

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SSIC NO. 5090.3

RESPONSE TO COMMENTS ON THE DRAFT ORDNANCE AND EXPLOSIVES WASTE/GEOTECHNICAL CHARACTERIZATION REPORT

PAGE 14 OF 14

FINAL ORDNANCE AND EXPLOSIVES WASTE/GEOTECHNICAL CHARACTERIZATION REPORT; ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION, TIME-CRITICAL REMOVAL ACTION, AND GEOTECHNICAL AND SEISMIC EVALUATIONS

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TABLE OF CONTENTS

PAGE

TABLE OF CONTENTS
(Continued)

TABLE OF CONTENTS
(Continued)

PAGE

TABLE OF CONTENTS
(Continued)

APPENDICES

ATTACHMENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF FIGURES
(Continued)

ABBREVIATIONS AND ACRONYMS

ARAR	applicable or relevant and appropriate requirement
ASTM	American Society for Testing and Materials
AT/AP	anti-tank/anti-personnel
bgs	below ground surface
bpf	blows per foot
BRAC	Base Realignment and Closure
BSSC	Building Seismic Safety Council
CAD	computer-assisted drawing
Caltrans	California Department of Transportation
Cc	clay content
CCR	California Code of Regulations
CCS	California Coordinate System
CD	consolidated-drained
CDFG	California Division of Fish and Game
CDMG	California Division of Mines and Geology
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act
CFR	Code of Federal Regulations
CH	fat clay
CIWMB	California Integrated Waste Management Board
CL	sandy clay/silty clay
COC	chain-of-custody
CPT	cone penetration test
CQC	Contractor Quality Control
CQM	Construction Quality Management
CRR	cycling resistance ratio
CSR	cyclic stress ratio
CTO	Contract Task Order
CU	consolidated-undrained
DERP	Defense Environmental Restoration Program
DFW	definable feature of work

ABBREVIATIONS AND ACRONYMS
(Continued)

DGPS	digital global positioning system
DMM	discarded military munitions
DO	Delivery Order
DoD	Department of Defense
DQO	data quality objective
DTSC	Department of Toxic Substances Control
E&E	Ecology and Environment, Inc.
ECM	Environmental Compliance Manager
EcoSystems	EcoSystems Management Associates, Inc.
EFANW	Engineering Field Activities Northwest
EOD	Explosives Ordnance Disposal
EPA	U.S. Environmental Protection Agency
EQ	earthquake
ERA	ecological risk assessment
ESRP	Explosive Safety Remediation Plan
EZ	exclusion zone
FAC	facultative vegetation
FACW	facultative wetland vegetation
FC	fines content
FCR	Field Change Request
FEMA 273	Federal Emergency Management Agency 273
FS	ratio of CRR to CSR
ft/sec	feet per second
ft/sec^2	feet per second squared
FWENC	Foster Wheeler Environmental Corporation
g	acceleration due to gravity 32.2 ft/sec^2
GIS	Geographic Information System
HAI	Hushmand Associates, Inc.
HEI	high-explosive incendiary
HFA	Holguin, Fahan, & Associates, Inc.

ABBREVIATIONS AND ACRONYMS
(Continued)

HRG	Habitat Restoration Group
IBD	Inhabited Building Distances
I_c	soil behavior type index
in^2/sec	square inches per second
IR	Installation Restoration
IRP	Installation Restoration Program
kcf	kips per cubic foot
kHz	kilohertz
km	kilometer
KSR	Kister, Savio, and Rei, Inc.
K_y	yield acceleration
lbs	pounds
LPF	low plasticity fines
M	magnitude
MBTA	Migratory Bird Treaty Act
MC	Modified California
MCE	Maximum Credible Earthquake
MH	high plasticity
Mk	mark
M_L	local magnitude
ML	sandy silts/silty clays
mm	millimeter
mm/yr	millimeters per year
MMI	Modified Mercalli Intensity
MMR	Military Munitions Rule
MPE	Maximum Probable Earthquake
MPM	most probable munition
M_S	surface wave magnitude scale
msl	mean sea level
M_W	moment magnitude scale

ABBREVIATIONS AND ACRONYMS

(Continued)

MWF	Magnitude Weighting Factor
N/A	not applicable
NAD	North American Datum
NAVFAFC	Naval Facilities Engineering Command
NAVSEA	Naval Sea Systems Command
NC	Normally Consolidated
NCEER	National Center for Earthquake Engineering Research
NEHRP	National Earthquake Hazard Reduction Program
NEIC	National Earthquake Information Center
NEW	net explosive weight
NGS	National Geodetic Survey
NGVD	National Geodetic Vertical Datum
NOSSA	Naval Ordnance Safety and Sescurity Activity
NSRS	National Spatial Reference System
OBL	obligate wetland vegetation
OEW	Ordnance and Explosives Waste
OU	Operable Unit
PCB	polychlorinated biphenyl
pcf	pounds per cubic foot
PD	probability of detection
PEA	Pacific Engineering and Analysis
PES	potential explosion site
PHGA	peak horizontal ground acceleration
PRC	PRC Environmental Management, Inc.
PQCM	Project Quality Control Manager
psf	pounds per square foot
PTRD	Public Transportation Route Distances
$(q_{c1N})_{cs}$	"clean-sand equivalent" normalized soil penetration resistance
Q/D	Quantity Distance
QA	quality assurance

ABBREVIATIONS AND ACRONYMS
(Continued)

Qc	cone penetration tip resistance
QC	quality control
RAB	Restoration Advisory Board
RAC	Remedial Action Contract
RI	Remedial Investigation
rl-ss	right lateral, strike slip
ROICC	Resident Officer in Charge of Construction
RPM	Remedial Project Manager
RWQCB	Regional Water Quality Control Board
SC	clayey sand
SCEC	Southern California Earthquake Center
sec	seconds
SEP	Search and Effectiveness Probability
SFOBB project	SFOBB East Span Seismic Safety project
SFOBB	San Francisco-Oakland Bay Bridge
SHANSEP	Stress History and Normalized Engineering Properties
SM	silty sand
SOP	Standard Operating Procedure
SOW	Scope of Work
SP	poorly graded sand
SPT	standard penetration test
SPT-N	standard penetration test blow count
SSPORTS	Supervisor of Shipbuilding, Conversion and Repair, Portsmouth
Std.	standard
SUXOS	Senior UXO Supervisor
SWAT	Solid Waste Assessment Test
SWDIV	Southwest Division Naval Facilities Engineering Command
SWRCB	State Water Resources Control Board
TBC	to be considered
TCRA	Time-Critical Removal Action

ABBREVIATIONS AND ACRONYMS

(Continued)

Teratest	Teratest Labs, Inc.
tsf	tons per square feet
TtEMI	Tetra Tech EM, Inc.
TTLC	Total Threshold Limit Concentration
UC	under-consolidated
UFGS	Unified Facilities Guide Specifications
USACE	United States Army Corps of Engineers
USFWS	United States Fish and Wildlife Service
USGS	United States Geological Survey
UU	unconsolidated-undrained
UXO	unexploded ordnance
Vs	shear-wave velocity
WDR	Waste Discharge Requirement
WET	wetland evaluation technique
WG99	1999 Working Group
WMM	waste military munitions
Work Plan	Final Focused Remedial Investigation Work Plan

EXECUTIVE SUMMARY

This Ordnance and Explosives Waste (OEW)/Geotechnical Characterization Report consists of an OEW characterization and geotechnical and seismic evaluations at Installation Restoration (IR) Site 2 in Operable Unit (OU)-4A of Alameda Point, Alameda, California. The scope of the OEW characterization included location, identification, and removal of any OEW found on the ground surface of the site in order to safely perform the geotechnical and seismic evaluation field tasks and for future grading operations. The geotechnical and seismic evaluations were conducted to characterize the existing soil covers, identify seismic hazards, and perform preliminary engineering analyses. In addition to the work performed at IR Sites 1 and 2, geotechnical and seismic evaluations were also conducted for an area between IR Sites 1 and 2 (the Additional Investigation Area). The additional investigation was needed to address variable subsurface site conditions encountered.

Prior to conducting any field activities, a visual reconnaissance and surface sweep of all support zones, staging areas, and access roads were conducted by unexploded ordnance (UXO) technicians to remove any potentially dangerous OEW from the ground surface. Vegetation was cut to a height of no more than 4 inches to facilitate surface OEW characterization of the entire site. During the surface characterization of IR Site 2, one anti-tank/anti-personnel (AT/AP) inert land mine and one 20 millimeter (mm) target practice projectile were found. In addition to the surface characterization activities, a Time-Critical Removal Action (TCRA) was performed at the Possible OEW Burial Site, a 2.3-acre area located at the southern part of IR Site 2. A complete discussion of the TCRA is provided in a separate *Final Time-Critical Removal Action Closeout Report* [Foster Wheeler Environmental Corporation (FWENC, 2002a)]. During the TCRA, 8,675 20mm target practice projectiles were uncovered. None of the OEW encountered contained any explosives or energetics. The AT/AP inert land mine was turned over to the Navy Explosive Ordnance Disposal (EOD) personnel. All of the target practice projectiles found were demilitarized and shipped to a Class III landfill facility for disposal as non-hazardous scrap steel.

Surveying activities conducted in support of field activities included UXO grid survey, survey for the TCRA area, survey of geotechnical sample locations, and bathymetric and shoreline surveys. Kister, Savio, and Rei, Inc. (KSR), a California-licensed land surveyor, established control points for IR Site 2. After OEW characterization activities were complete, KSR surveyed the proposed cone penetration test (CPT), boring, and test pit locations identified in the *Final Focused Remedial Investigation Work Plan* (Work Plan) (FWENC, 2002b). A bathymetric survey and shoreline survey were conducted by EcoSystems Management Associates, Inc. (EcoSystems) in January 2002. The bathymetric survey extended from the high water mark to 500 feet offshore. Survey lines were established normal to the general shoreline orientation at 50-foot intervals. Tie lines were set up to intersect the survey lines at approximately 100-foot spacing from the shoreline to the offshore limit of the survey area. The shoreline survey consisted of surveying the shoreline's horizontal location.

Three major tasks were performed as part of the geotechnical and seismic evaluations. These included: 1) collection of soil samples/field data, 2) laboratory geotechnical soil testing, and 3) geotechnical and seismic hazard evaluation. Soil sampling activities included site preparation, metal avoidance activities, excavating test pits, cone penetration testing, drilling boreholes, collecting soil samples, and processing samples (storing, recording, and transporting soil samples). Testing of soil samples was conducted by Teratest Labs, Inc. (Teratest). Geotechnical and seismic evaluations included calculating liquefaction potential, estimating static- and seismic-related settlement and lateral displacements, and performing static and seismic slope stability analyses.

The results obtained from the field exploration and laboratory geotechnical soil testing were used to evaluate geotechnical characteristics of the existing soil cover and underlying soil layers, calculate immediate and long-term settlements from a proposed landfill cover, estimate seismically induced settlements and lateral displacements, and perform static and dynamic stability analyses of various slopes at the site.

A geotechnical evaluation was conducted in the immediate offshore and upland area of IR Site 2, including the Additional Investigation Area. The field investigations conducted to collect this data included 21 CPTs, 12 test pit explorations, and 15 soil borings (nine upland and six offshore). Representative disturbed and relatively undisturbed soil samples were retrieved for geotechnical testing. No chemical analyses were performed. Soil boring and test pit logs were recorded and used to characterize subsurface conditions at the site. Based on the test pit exploration, the existing soil cover was found to be inconsistent and poorly compacted. Therefore, the material was determined to be unsuitable for use as part of the final cover design. The maximum ground settlement expected to occur from an assumed 4-foot landfill cap is approximately 13 inches. Higher overall and difference in total settlements could occur in areas where additional fill material for grading will be placed. However, these settlements are expected to occur over a long period of time (40 years or more). Therefore, settlements do not pose an immediate hazard. Additional evaluations can be performed in specific areas of concern as part of the final cover design.

Different cross sections at the site were analyzed for static (pre-earthquake) stability. The program, PC-STABL-5M, based on limit equilibrium theory, was used to obtain factors of safety against slope failure (Achilleos, 1988). This factor is defined as the ratio of resisting (stabilizing) forces to the driving forces trying to displace the slope. Guidelines for the stability analyses are provided in Title 27 California Code of Regulations (CCR). However, no specific value for the static factor of safety is provided. The current state of practice in California is to require a minimum static factor of safety of 1.5. Six different cross sections across IR Site 2 (Cross

Sections C-C' to H-H') and one in the Additional Investigation Area (Section I-I') were analyzed. Cross sections at IR Site 2 were analyzed with an assumed 4-foot-thick soil cover. Cross section I-I' is located on a former air strip, where no future soil is planned. All cross sections analyzed (except Cross Section C-C') had static factors of safety greater than 1.5. The factor of safety calculated for Section C-C' was 1.46, less than the minimum required by the state of California. Therefore, remedial measures involving geotechnical improvements of existing site conditions are needed to increase the static factors of safety to meet the current standard of practice in California.

Seismic hazards identified at IR Site 2 included liquefaction potential and seismic slope instability. An extensive seismic hazard analysis was performed by Hushmand Associates, Inc. (HAI) to obtain the site design earthquake motions [peak horizontal ground accelerations (PHGAs), site design response spectras, and representative acceleration time histories] and to estimate seismically induced ground deformations. An integrated CPT-based method (Robertson and Wride, 1997) was used to quantify the potential for liquefaction and identify areas susceptible to liquefaction. Based on the analyses, the upper fill material consisting mainly of dredged soils from San Francisco Bay had a high potential for liquefaction and was designated as liquefiable. The upper fill material was classified in accordance with the United Soil Classification System as very loose to medium dense sand with occasional layers of fine-grained soils and gravel. Liquefaction-induced settlements in the fill layer are estimated to be up to 12 inches. In addition to liquefaction-induced settlements from the upper fill material, soil sediments from the Young Bay Mud layer (below the fill material) could experience approximately 4 to 6 inches of settlement due to liquefaction and consolidation. Therefore, the total seismically induced settlements could be as high as 18 inches (1.5 feet). In addition, using empirical methods, horizontal displacements toward San Francisco Bay due to liquefaction of the upper fill layer were estimated at more than 20 feet. Using Newmark-type (Makdisi and Seed, 1978) procedures, permanent lateral displacements at the site were obtained. Based on preliminary findings, predicted seismically induced slope deformations are high, ranging from 4 to 19 feet. For post-earthquake stability conditions, the United States Army Corps of Engineers (USACE) recommends a post-earthquake static factor of safety greater than 1.0. This criterion was satisfied for all cross sections, except Cross Section F-F'.

All necessary precautions were taken in the field investigation activities to mitigate impact to the existing wetland and nesting environments at IR Site 2. For example, after completion of the aforementioned field activities, it was determined that the Possible OEW Burial Site area created a potential nesting habitat similar to that preferred by the California least terns. This condition was considered unacceptable because of the presence of the feral cat and raptor (American Peregrine falcons, red-tailed hawks) populations that are already established there. To remedy the situation, a 12-mil-thick high-strength polyethyelene liner was installed over the entire Possible OEW Burial Site area to deter the California least terns from nesting in that area. The liner was removed at the conclusion of the nesting season and the area was hydroseeded.

As the lead agency for the environmental Installation Restoration Program (IRP) activities at Alameda Point, the Navy is responsible for community relation activities. The proposed IR Site 2 project activities were discussed with the Restoration Advisory Board (RAB) that includes interested community members and representatives from regulatory agencies.

For a complete record of activities conducted at IR Site 2, documents have been compiled and are contained in the information repositories that are located at:

1. Alameda Main Public Library (Historic Alameda High School)
   2220 Central Avenue
   Alameda, California
2. Alameda Point
   950 West Mall Square, Suite 141
   Alameda, California

The complete Administrative Record is located at 1220 Pacific Highway, San Diego, California, and is maintained by Ms. Diana Silva, Southwest Division Naval Facilities Engineering Command (SWDIV) Administration Record Manager at (619) 532-3676.

1.0 INTRODUCTION

The Southwest Division Naval Facilities Engineering Command (SWDIV) authorized Foster Wheeler Environmental Corporation (FWENC) to perform an ordnance and explosives waste (OEW) characterization, Time-Critical Removal Action (TCRA), and geotechnical and seismic evaluations of the former solid waste disposal site identified as Installation Restoration (IR) Site 2, Operable Unit (OU)-4A of Alameda Point, Alameda, California (Figure 1-1). The TCRA activities are addressed in a separate *Final Time-Critical Removal Action Closeout Report* (FWENC, 2002a). This work is part of an ongoing focused Remedial Investigation (RI) performed by FWENC that includes an OEW characterization and geotechnical and seismic evaluation for IR Site 1 (OU-3), an area just north of IR Site 2. In addition to the work performed at IR Sites 1 and 2, geotechnical and seismic evaluations were also conducted for an area between IR Sites 1 and 2 (the Additional Investigation Area). The findings of the geotechnical and seismic investigation performed at IR Site 1 were considered in this report as part of our evaluation of IR Site 2 and the Additional Investigation Area. The OEW characterization and geotechnical and seismic evaluations performed were a component of the Navy's RI/Feasibility Study of the site under the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA), more widely known as "Superfund."

The authorization for this work was originally issued under Engineering Field Activities Northwest (EFANW) Remedial Action Contract (RAC) II No. N44255-95-D-6030, Delivery Order (DO) No. 0095, under the Defense Environmental Restoration Program (DERP) for Base Realignment and Closure (BRAC). The performance period under the current contract expired on September 30, 2002, the close of the federal fiscal accounting period. A new Contract Task Order (CTO) describing the current RI work under a revised Scope of Work (SOW) was issued under RAC N68711-98-D-5713. The new CTO, No. 0054, authorizes FWENC to complete all remaining work originally authorized under DO No. 0095.

1.1 BACKGROUND

IR Site 2 is located on the western coastline of Alameda Point, Alameda, California, and includes the West Beach Landfill (the landfill), the West Beach Landfill Wetland (the wetland), and the associated interior and coastal margins (Figure 1-2).

Alameda Point is located on the westernmost end of Alameda Island, which lies on the eastern side of San Francisco Bay, adjacent to the city of Oakland. Alameda Point is rectangular in shape, approximately 2 miles long east-to-west, 1 mile wide north-to-south, and was occupied by the 1,734-acre Alameda Point until its closure in 1997.

IR Site 1, a waste disposal area used between 1943 to 1956, is located just north of IR Site 2 (see Figure 1-2). In between IR Sites 1 and 2, a narrow strip of land separates the two sites and was

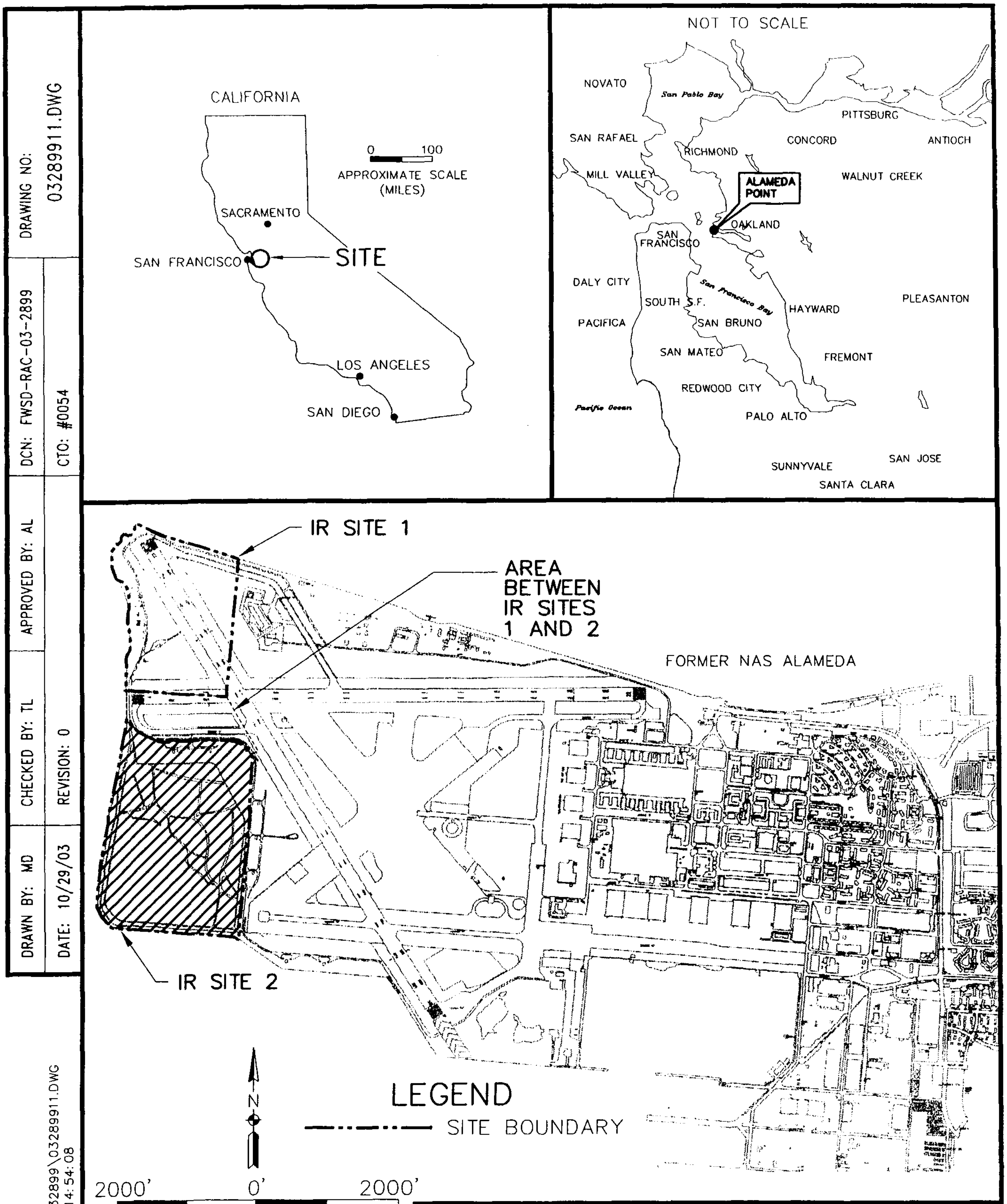
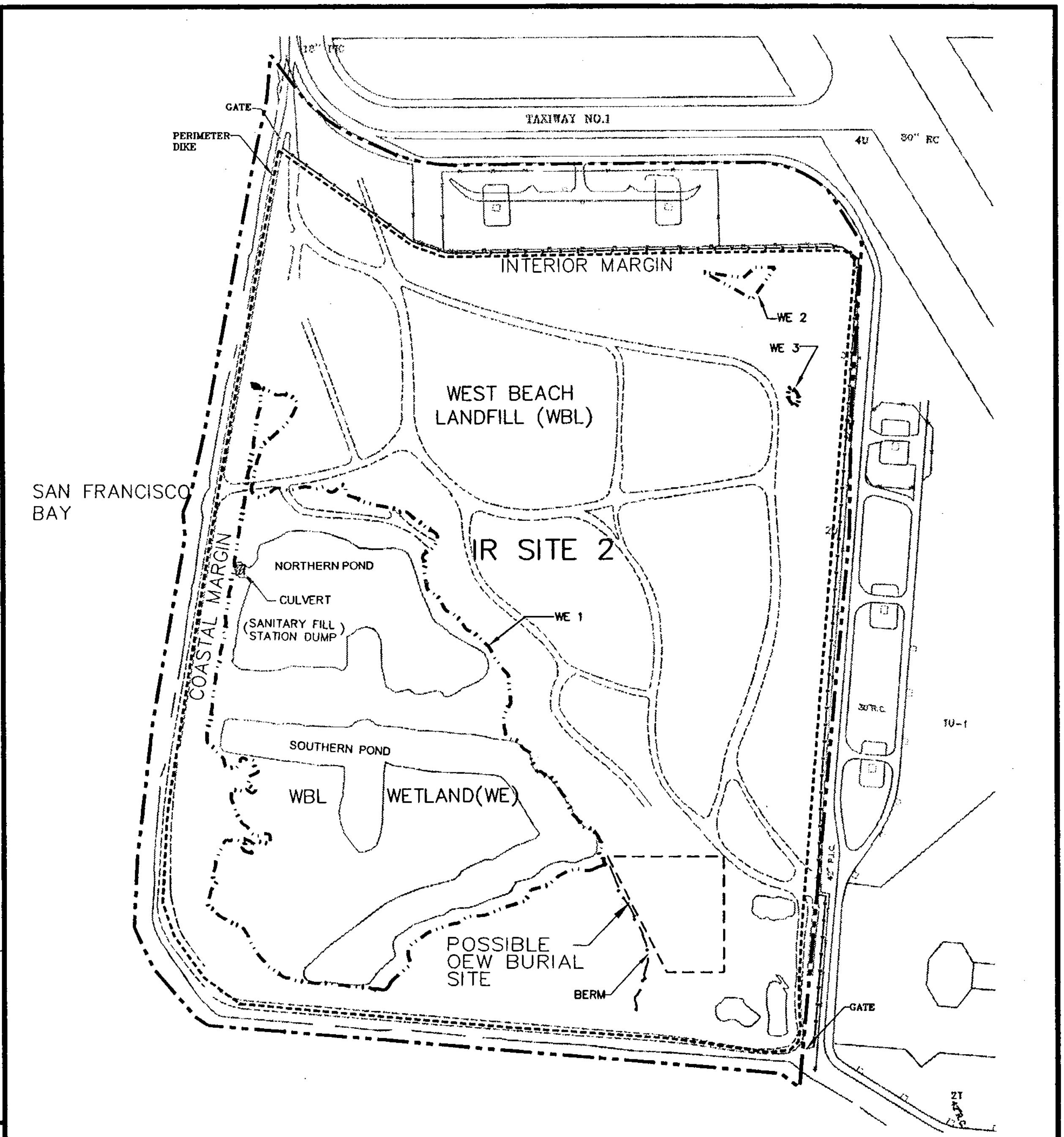


Figure 1-1
ALAMEDA POINT VICINITY MAP
ALAMEDA, CA

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION



LEGEND

- SITE BOUNDARY
- FENCELINE
- WETLAND BOUNDARY
- PERIMETER DIKE
- BERM
- WE — WETLAND

N

500 250 0 500

SCALE IN FEET

Figure 1-2
IR SITE 2 SITE PLAN
ALAMEDA, CA

Southwest Division
Naval Facilities Engineering Command

TETRA TECH FW, INC.

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formerly used as a runway. This area, identified in this report as the Additional Investigation Area, was investigated as part of the RI work since it can impact the findings of this investigation. Further discussion of the work performed in the Additional Investigation Area is presented in Section 1.3.3.

1.1.1 Site Description

IR Site 2 encompasses approximately 110 acres and is bordered by San Francisco Bay to the south and west. The landfill at IR Site 2 covers approximately 77 acres in the extreme southwestern portion of Alameda Point. The wetland covers approximately 30 acres and is bounded by the landfill to the north and east and by the coastal margin adjacent to the San Francisco Bay on the south and west. The wetland contains two perennial ponds. The northern pond is connected to the bay by a culvert. The southern pond was created by removal of dredged materials for use as landfill cover. Fresh water has since filled the excavation area and created the pond. The only material known to have been deposited in the wetland is scrap metal [Ecology and Environment, Inc. (E&E), 1983)].

A thin strip of land between the landfill or wetland and the bay is referred to as the coastal margin. It acts as a buffer for the landfill and the wetland and is composed of the perimeter dike and riprap seawall. Subsurface materials in the coastal margin differ from those in the landfill and wetland. The interior margin lies outside the landfill and wetland, to the north and east. It also contains part of the perimeter dike and includes all areas outside the dike to the north and east. It is a geographic definition used primarily for classifying sampling locations. Grasses and thistles are the dominant vegetation of the upland areas while seaside trefoil, brass buttons, and pickleweed inhabit the wetlands [United States Fish and Wildlife Service (USFWS), 1998].

1.1.2 Site History

IR Site 2 was used as the main disposal area for the Alameda Point from approximately 1952 through 1978. An estimated 1.6-million tons of waste were deposited (E&E, 1983). The wastes included municipal solid waste, waste chemical drums (contents unknown), solvents, oily waste and sludge, paint waste, plating wastes, industrial strippers and cleaners, acids, mercury, polychlorinated biphenyl (PCB)-containing liquids, batteries, low-level radiological waste from radium dials and dial painting, scrap metal, inert ordnance, asbestos, several pesticides (solid and liquid), tear gas agent, biological waste from the Oak Knoll Naval Hospital, creosote, dredge spoils, and waste medicines and reagents (E&E, 1983). OEW may have also been deposited in the 2.5-acre (approximate) Possible OEW Burial Site located in the southern part of the landfill. A seawall was constructed along the southern and western edges of the site, and a 36-inch culvert was installed in the seawall to hydraulically connect San Francisco Bay to waters within the seawall. A substantial (10- to 15-foot) dike was installed around the perimeter of the site when disposal operations ceased.

An Emergency Removal Action to remove live 20mm high-explosive projectiles was completed in 1998 on IR Site 1, which is immediately adjacent to IR Site 2. The projectiles were discovered on the ground and in shallow pits on the former small range during the course of a radiological survey being conducted on Site 1. Because of the projectiles' location, and the fact that the small arms range was constructed after the Site 1 landfill had been closed and capped, the projectiles were probably discarded there after the landfill was closed. The potential to encounter other discarded military munitions (DMM) on IR Site 1, IR Site 2, or elsewhere on Alameda Point cannot be disregarded.

1.1.3 Previous Investigations

A variety of investigations were completed to characterize the landfill and the wetland. The sampling was done to characterize the environmental media, hydrology, vegetation, and wildlife. Bioaccumulation testing, bioassays, and tissue sampling were also conducted to develop an ecological risk assessment (ERA). A summary of the investigations is provided as follows:

- Phases 1 and 2A Solid Waste Assessment Test (SWAT) investigation conducted in 1990
- Phases 5 and 6 SWAT investigation conducted in 1991
- ERA conducted in 1993
- Wetland delineation and wetland evaluation technique (WET) analyses conducted in 1993
- Additional field activities conducted in 1994 and 1995 in support of the ERA
- Threatened and endangered species survey conducted from 1995 to 1997
- Follow-up ecological investigation conducted in 1996 and 1997
- Groundwater monitoring conducted from 1991 to 1998
- Biological sampling conducted in 1998 to support the ERA
- Geophysical survey of the Possible OEW Burial Site in 1998

Soil Sampling

Surface and subsurface soil sampling activities at the landfill, the wetland, and the coastal margin occurred from 1990 through 1995 to provide information on the potential impacts of past disposal practices on soil and chemical characteristics. The upper 2 feet of subsurface soil at the landfill was sampled to determine if chemicals were present, and if so, to determine the lateral extent of their migration.

Groundwater Sampling

A total of 42 sampling wells were installed on IR Site 2 as a part of a groundwater characterization that was conducted to determine if any chemicals in the landfill were seeping into the groundwater and migrating off site. During the first investigation conducted in 1991 and 1992, 132 samples were taken from 29 monitoring wells during three sampling events. During the 1994–1995 investigations, 100 samples were taken from 12 locations during four sampling events.

Pond Water Sampling

Sediment pore waters were sampled to determine if chemicals were present in the pond sediments, and if so, if they were desorbing from the sediments and diffusing into the surface water, which could lead to adverse ecological effects in the benthic community. Sediment pond water was collected from three locations in the northern pond in 1996 and three locations in the southern pond in 1997.

Biotic Sampling

Tissue sampling was conducted in 1996 and 1998 to support the ERA and to estimate the potential chemical doses (if present) to upper-level trophic receptors. A sampling plan was developed to collect plants, invertebrates, non-migratory fishes, and small mammals from the wetland habitats, and plants, invertebrates, and small mammals from the terrestrial habitats of the landfill.

Threatened and Endangered Species Survey

An endangered (or threatened) species survey was conducted for the Navy from 1996-1997 by Tetra Tech EM, Inc. (TtEMI) to determine the occurrence of threatened or endangered species on Alameda Point. The survey included both literature reviews and field surveys and was conducted for plants, mammals, amphibians, reptiles, and birds.

A literature review conducted by the USFWS (USFWS, 1998) identified several threatened or endangered species of plants and animals that could occur on IR Site 2 given their presence on similar sites in the area, but none of them are known to currently inhabit IR Site 2. Threatened or endangered bird species that have been observed near the wetland on Alameda Point include the American peregrine falcon, western snowy plover, California least tern, salt marsh common yellowthroat, Alameda song sparrow, and California brown pelican. California least tern nests exist east of IR Site 2. All of the birds (except for the California brown pelican) could appear in IR Site 2, but none have been observed in recent years (USFWS, 1998).

Plant Survey

Field surveys to identify and document the presence of any threatened, endangered, or sensitive terrestrial plant species were performed on IR Site 2 as a part of the threatened or endangered species survey in 1997. Information from the plant surveys was used to help characterize the habitat at the landfill and wetland.

Benthic-Invertebrate Survey

Wetland sediment analyses were conducted in 1993 and 1994 to determine whether chemicals present in the wetland were impacting the benthic-invertebrate community structure or diversity. The samples were collected from four locations in the wetland, then sieved, and the invertebrates were identified and cataloged to characterize the community.

Avian Survey

Avian surveys were conducted at the wetland between January and May 1997 to characterize bird communities at IR Site 2 and to provide information for the selection of receptors for the ERA. The wetland was surveyed bimonthly (approximately) and a total of ten surveys were completed.

Toxicity Tests

Toxicity tests of wetland sediments were conducted in 1993 and 1994 at seven locations in the wetland. The solid-phase toxicity tests were conducted from samples at five locations in the northern pond and two locations in the southern pond. Additionally, five replicate tests were conducted for the amphipod and polychaete worm in surface sediments at each of the seven sample locations.

Bioaccumulation Test

Bioaccumulation tests using the clam (*Macoma nausta)* and the sea urchin (*Strongylocentrotus purpuratus*) were conducted from wetland sediments in 1993 and 1994. The samples were taken from four locations in the wetland and were used to determine if chemicals sorbed in the sediments were bioavailable to benthic organisms and could potentially bioaccumulate up the food chain.

Radiation Survey

Several radiological surveys were conducted on IR Site 2 because of the possibility that wastes from the radium dial painting shop that operated on Alameda Point had been discarded in the landfill. PRC Environmental Management, Inc. (PRC) conducted a near-surface radiological scoping survey of the accessible landfill areas in 1995, and additional surveys from May to September 1996 (PRC, 1997). A total of 40 radiological anomalies were discovered during the surveys. Supervisor of Shipbuilding, Conversion and Repair, Portsmouth (SSPORTS) Environmental Detachment conducted a more comprehensive radiological survey in 1998 and 1999 (SSPORTS, 1998; SSPORTS, 1999), which found 951 points with radiation counts greater than the defined threshold of twice the normal background level. Removal actions were completed at 51 sites with radiation counts over four times the normal background level.

Cone Penetration Test

Cone penetration test (CPT) surveys were conducted in 1994 at seven locations in the landfill as part of a larger effort to characterize the lithology of Alameda Point.

Wetland Delineation and Wetland Evaluation Technique Analysis

In February and March 1993, the IR Site 2 jurisdictional wetland was delineated and an analysis of the wetland using WET was completed in March 1993. Habitat Restoration Group (HRG) documented the work in the following reports: *Naval Air Station Alameda Preliminary Wetland Delineation* (HRG, 1993a) and *Naval Air Station Alameda WET Analysis* (HRG, 1993b).

Geophysical Survey

A 2.5-acre (approximate) area, the Possible OEW Burial Site, in the southeast corner of the landfill at IR Site 2, was identified by SSPORTS unexploded ordnance (UXO) personnel as a possible ordnance burial site. The identification of this site was based on the results of a geophysical survey of the area, the previous use of the site, and interviews conducted with Alameda Point Weapons Department personnel. Attempts to discriminate several large, subsurface masses and anomalies as ordnance or construction debris/waste were unsuccessful due to the high background noise of the area and the large amount of debris present. Information from survey results, personnel interviews, and archive data indicate that the area was once used as a burial site for inert ordnance and that buried OEW/UXO may be present at the site (SSPORTS, 1999).

A summary of the types of investigations performed and the particular medium investigated in IR Site 2 is presented in Table 1-1.

1.2 PURPOSE AND OBJECTIVE

The objective of this action was to complete a surface OEW characterization and to complete geotechnical and seismic evaluations of IR Site 2, including the Additional Investigation Area, in accordance with the approved *Final Focused Remedial Investigation Work Plan* (Work Plan) (FWENC, 2002b). Findings of the investigation and evaluations will be incorporated into the RI and Feasibility Study Reports for IR Site 2. The results of the geotechnical and seismic evaluation will be used to identify associated hazards for the Feasibility Study.

The site is currently used as a bird and wildlife sanctuary and is proposed for transfer to the USFWS for use as a National Wildlife Refuge. An OEW characterization and removal of any OEW found has been completed, which is required prior to property transfer to the USFWS. The findings from the geotechnical and seismic evaluations will be used in the design and construction of the recommended remedial alternative.

TABLE 1-1

SUMMARY OF IR SITE 2 SAMPLE INVESTIGATIONS AND MEDIA

Investigation	Surface Soil	Subsurface Soil	Groundwater	Surface Water	Sediment	Tissue	Bioassays	Plant and Animal Surveys	Pore Water
SWAT Phases 1 and 2A (1990)	X	X							
SWAT Phases 5 and 6 (1991)	X	X		X	X				
Ecological assessment (1993)					X		X	X	
Ecological assessment (1994-1995)	X	X		X			X	X	
Threatened and endangered species survey (1995-1997)								X	
Follow-up ecological investigation (1996-1997)				X	X	X			X
Groundwater monitoring (1991-1998)			X						
Biological sampling (1998)						X			

Notes:

IR – Installation Restoration
SWAT – Solid Waste Assessment Test

1.3 SUMMARY OF WORK

The following tasks were performed as part of the SOW described in the Work Plan (FWENC, 2002b):

- Surface OEW Characterization
- Geotechnical Evaluation
- Seismic Evaluation
- Additional Investigation in Area Between IR Sites 1 and 2

Detailed descriptions of these activities are discussed in subsequent sections of this document. A brief description of the tasks is provided in the following sections.

1.3.1 OEW Characterization

OEW characterization activities included reviewing site information, qualifying the UXO technicians, performing an OEW sweep, moving identified OEW to Magazine M353, and demilitarizing recovered OEW items. Upon completion of the OEW surface characterization, UXO technicians assisted in removing metal debris that would potentially inhibit test pit excavations and boreholes activities.

Existing historical and archival site information was reviewed to conservatively estimate the most probable munition (MPM) likely to be encountered during characterization activities, assess the related hazards for the MPM, and develop safety precautions.

As established in the data quality objectives (DQOs) (see Section 1.4), the UXO characterization team was certified in the surface quality control (QC) test grid in accordance with Search and Effectiveness Probability (SEP) test parameters. Prior to any field activities, the UXO team conducted a surface sweep of all support areas. A 200-foot by 200-foot grid coordinate system was established, which was used to conduct surface OEW characterization activities. Each delineated grid was then characterized by certified UXO technicians with the locations of identified OEW marked on the site map. OEW was then examined to determine if it could be safely moved. Recovered OEW was stored in Magazine M353 until the completion of characterization activities, at which time, it was demilitarized.

Characterization methods and results, as well as metal avoidance procedures, are further discussed in Section 3.0, Ordnance and Explosives Waste Characterization.

1.3.2 Geotechnical and Seismic Evaluation

The three main tasks involving geotechnical and seismic evaluations included 1) collection of soil samples/field data, 2) geotechnical soil testing, and 3) seismic hazard evaluation. Soil sampling activities included site preparation, metal avoidance activities, excavating test pits, cone penetration testing, drilling boreholes, collecting soil samples, and sample processing (storing, recording, and transporting soil samples). Geotechnical soil testing was conducted by Teratest Labs, Inc. (Teratest). Geotechnical and seismic evaluations included calculation of liquefaction potential, estimation of static- and seismic-related settlement and permanent lateral displacement, and evaluation of static and seismic slope stability.

The results obtained from the field exploration and laboratory soil testing were used to evaluate the following:

- Geotechnical characteristics of existing soil cover and underlying soil layers
- Immediate and long-term settlements from placement of a landfill cap
- Seismically induced settlements and lateral displacements
- Static and dynamic stability of various shoreline slopes with and without placement of a landfill cap

1.3.2.1 Geotechnical Evaluation

The field investigations involved performing 21 CPTs, excavating 12 test pits, and drilling 15 soil borings (nine upland borings and six offshore borings) using a mud rotary system. Representative disturbed and relatively undisturbed soil samples were retrieved for geotechnical analyses. Standard penetration test (SPT) blow counts for granular soils were recorded for liquefaction evaluations. No chemical analyses were performed.

Immediate and long-term settlements at IR Site 2, due to future placement of a landfill cap, were estimated using the theory of elasticity and one-dimensional consolidation theory (Terzaghi as described by Coduto, 1994). Static stability of various slope cross sections of IR Site 2 and the Additional Investigation Area between IR Sites 1 and 2 was analyzed using the program, PC-STABL-5M (Achilleos, 1988), to obtain factors of safety against slope failure. The analyses are based on two-dimensional conventional limit equilibrium theory.

1.3.2.2 Seismic Evaluation

Field testing to determine static and dynamic soil parameters was conducted as the first step in the seismic evaluation process. A deterministic seismic shaking hazard evaluation was then performed to estimate site design ground motions [peak horizontal ground acceleration (PHGA), site design response spectra, and representative acceleration time histories] at IR Site 2. The evaluation considered seismicity of the region, nearby faults, attenuation relationships, and soil amplification.

Newmark-type deformation analysis methods (Makdisi and Seed, 1978) were used to estimate seismically induced slope deformations. A computer program, PC-STABL-5M, was used to perform pseudo-static analysis to obtain yield accelerations (the pseudo-static acceleration resulting in a factor of safety of 1.0, which is indicative of imminent slope movement) for different cross sections at the site (Achilleos, 1998). Permanent displacements were obtained by double integration of average acceleration time histories of potential sliding masses to estimate incremental slope movement whenever acceleration of sliding mass exceeded the yield acceleration.

Liquefaction potential evaluation was performed using integrated CPT-based (Robertson and Wride, 1997) and SPT-based (Youd and Idriss, 1997; Youd et al., 2002) procedures. Liquefaction-induced ground surface subsidence in areas away from perimeter slopes was estimated by calculating the cyclic stress ratio (CSR) profile from the site design PHGA, and cycling resistance ratio (CRR) from CPT or SPT data and correlating with ground settlements (Ishihara and Yoshimine, 1992). Empirical relations developed from a large case history data set of measured displacements for lateral spreads (Youd et al, 2002) were used to estimate liquefaction-induced lateral displacements.

1.3.3 Additional Investigation in Area Between IR Sites 1 and 2

An additional field investigation was conducted in an area between IR Sites 1 and 2 (the Additional Investigation Area) due to insufficient geological data. Previous investigations indicated the existence of a thick Bay Sediments layer in the northern part of IR Site 2 and in the area between IR Sites 1 and 2 (TtEMI, 1999). A geological cross section shows that the Bay Sediments layer extends from IR Site 1 to IR Site 2 and reaches a depth of up to 80 feet below ground surface (bgs) around the Additional Investigation Area (Geological Cross Section B-B', Appendix A). The Bay Sediments layer consists of a wide range of soil types including poorly graded sand (SP), silty sand (SM), clayey sand (SC), sandy silts/silty clays (ML), and sandy clay/silty clay (CL).

Results of a recent field exploration at IR Site 2 indicate that a weak soil layer exists, extending from 25 to 75 feet bgs at the northern tip of IR Site 2. This layer consists mostly of sensitive fine-grained material (Young Bay Mud, silty clay, and some loose sand). A CPT in this area shows cone resistance values less than 15 tons per square feet (tsf) at elevations between 25 to 75 feet bgs (C-2-15a, Appendix B). Also, blow counts from SPT and Modified California (MC) samplers were in the single digits (B-2-11, Appendix C). At IR Site 1, a weak Young Bay Mud layer was also present. However, it extended only down to 45 feet bgs and was underlain by a dense Merritt Sand layer extending to 90 feet bgs (FWENC, 2002a). The blow counts recorded for this Merritt Sand layer were consistently in the 30 blow count range with refusal (greater than 50 blow counts) encountered at several locations.

The results of the field investigation at IR Site 2 confirmed the depth of the Bay Sediments layer and indicated that this layer has low shear strength. The depth and strength of the Bay Sediments layer will impact slope stability at IR Site 2. Since the Bay Sediments layer exists in the

Additional Investigative Area and extends as deep or deeper than at IR Site 2, it is expected that factors of safety against slope failure will be lower in this area. Therefore, potential for slope failure would be higher in these areas. Any remedial alternatives proposed to mitigate geotechnical and seismic hazards will also be affected by soil conditions in this area. Since there is wide variability in the reported soil types for the Bay Sediments layer, additional field explorations were performed to better delineate the properties of the Bay Sediments layer.

Six CPTs and four soil borings were performed at the Additional Investigation Area. Findings from the field exploration and subsequent testing of soil samples obtained from the soil borings were used to determine long-term and seismic stability at this area.

1.4 DATA QUALITY OBJECTIVES PROCESS

The 7-Step DQO process was used to evaluate the scientific data collection elements of the Work Plan (FWENC, 2002b). The process consists of the following steps as defined in the *Guidance for the Data Quality Objectives Process* [U.S. Environmental Protection Agency (EPA), 1994]:

Step 1: State the Problem – includes identifying members of the planning team, identifying the primary decision maker of the planning team and defining each member's role and responsibility during the DQO process, developing a concise description of the problem, and specifying the available resources and relevant deadlines for the study.

Step 2: Identify the Decision – includes identifying the principal study question, defining the alternative actions that could result from resolution of the principal study question, combining the principal study question and the alternative actions into a decision statement, and organizing multiple decisions.

Step 3: Identify Inputs to the Decision – includes identifying the information that will be required to resolve the decision statement, determining the sources for each item of information identified, identifying the information that is needed to establish the action level, and confirming that appropriate analytical methods exist to provide the necessary data.

Step 4: Define the Study Boundaries – includes specifying the characteristics that define the population of interest, defining the spatial boundary of the decision statement, defining the temporal boundary of the problem, defining the scale of decision making, and identifying practical constraints on data collection.

Step 5: Develop a Decision Rule – includes specifying the statistical parameter that characterizes the population (parameter of interest), specifying the action level for the study, and developing a decision rule.

Step 6: Specify Limits on Decision Errors – includes determining the possible range of the parameter of interest, identifying the decision errors and choosing the null hypothesis; specifying a range of possible parameter values where the consequences of decision errors are relatively minor (gray region), and assigning probability limits to points above and below the gray region that reflect the tolerable probability for the occurrence of decision errors.

Step 7: Optimize the Design for Obtaining Data – includes reviewing the DQO outputs and existing environmental data, developing general data collection design alternatives, formulating the mathematical expressions needed to solve the design problems for each data collection design alternative, selecting the optimal sample size that satisfies the DQOs for each data collection design alternative, selecting the most resource-effective data collection design that satisfies all of the DQOs, and documenting the operational details and theoretical assumptions of the selected design in the Sampling and Analysis Plan.

These steps were used to analyze both the OEW characterization and geotechnical and seismic characterization aspects of the project. Tables 1-2 and 1-3 summarize each step of the DQO process for OEW characterization and geotechnical and seismic characterization, respectively.

Additional details regarding implementation of Step 7, pertaining to OEW and geotechnical activities, are described below:

OEW

- **UXO technicians will establish a Cartesian coordinate search grid** – The UXO technicians established a 200-foot by 200-foot grid coordinate system that was used to conduct surface OEW characterization activities.
- **UXO technicians will complete a surface sweep** – the UXO characterization team was certified in the surface QC test grid in accordance with SEP test parameters and then performed the surface sweep of IR Site 2.
- **Optimized process for packing, certifying, and shipping OEW** – this process was optimized by designating a specialized subcontractor to handle this aspect of the work. No OEW was encountered that required this process to be implemented.

Additional details regarding the search, grid and surface sweep are included in Section 3.0 (Ordnance and Explosives Waste Characterization).

Geotechnical

- **Upland samples will be collected to a minimum of a 60-foot depth** – all borings and CPTs were advanced to a minimum 60-foot depth.
- **Samples will be collected every 5 to 10 feet or at any change of formation based on the results of previous CPT and field geologist/engineer observations** – samples were collected at approximately 5- to 10-foot intervals. Results of CPTs, performed prior to drilling, dictated where samples were collected. In general, samples were collected every 5 to 10 feet where fill material and weak clay layers were observed. In general, Shelby tube samples were collected on soft, fine-grained (clayey) soils, and MC and SPT samples were collected on coarse-grained (sandy) soils.
- **Similarly, the sample quantity and laboratory testing program will be refined based on the past field test results** – the number of samples analyzed was reduced by reviewing available past geotechnical field test results (TtEMI, 1999; 2001).

TABLE 1-2

DATA QUALITY OBJECTIVES FOR ORDNANCE AND EXPLOSIVES CONCERNS

STEP 1	STEP 2	STEP 3	STEP 4	STEP 5	STEP 6	STEP 7
Statement of Problem	**Decisions**	**Input to the Decisions**	**Boundaries of the Study**	**Decision Rules**	**Limits on Decision Errors**	**Optimizing the Design**
Spent OEW/UXO may have been buried in the landfill portion of IR Site 2. OEW was found on adjacent IR Site 1 during a previous survey. Site must be investigated to determine if OEW contamination exists. Site must be clear prior to land transfer. IR Site 2 was once a landfill where metal debris was buried. No live OEW is expected to be encountered.	Is surface and subsurface OEW contamination likely? What procedures will be used for OEW that is not safe to move? What procedures will be used for OEW that can be shipped?	UXO Site Investigation by SSPORTS (1999). *Initial Assessment Study of Naval Air Station, Alameda, California, Final Report* (E&E, 1983). Results of the planned surface sweep. OEW safety, packaging, and shipping publications. SEP test parameters as described in SOP-1 in the Work Plan (FWENC, 2002b).	IR Site 2, OU-4A of the Alameda Point. Surface sweep of entire site, excavation of Possible OEW Burial Site. Area of surface sweep is described in Figure 2-1 in the Work Plan (FWENC, 2002b). Nesting season of listed species may affect demobilization date. Federal and state regulations affect the packing, transportation, and treatment of OEW. CQC Plan (FWENC, 2002b) (SEP procedures) affect and quantify sweep procedures.	If no OEW is encountered during the surface and subsurface investigation, then no further action concerning OEW will be taken. If OEW is encountered, it will be considered investigation-derived waste and treated according to its status (safe, unsafe).	SEP tests will ensure 90 percent confidence level for sweep effectiveness. SEP tests will measure detection probability. If SEP tests results fall below 85 percent, then corrective measures outlined in CQC Plan (FWENC, 2002b) will be taken. OEW encountered will be evaluated as follows: - If unsafe to ship, a military EOD unit will respond. - If safe to ship, OEW will be packed and shipped in accordance with existing regulations and procedures.	Surveyors will establish control for the installation of a Cartesian coordinate search grid. UXO technicians will complete surface sweep and the subsurface excavation. Process for packing, certifying, and shipping OEW optimized. Process for certifying UXO sweep team in place. All OEW will be counted, photographed, and logged.

Notes:

CQC – Contractor Quality Control
E&E – Ecology and Environmental, Inc.
EOD – Explosive Ordnance Disposal
FWENC – Foster Wheeler Environmental Corporation
IR – Installation Restoration
OEW – ordnance and explosive waste
OU – Operable Unit
SEP – Search and Effectiveness Probability
SOP – Standard Operating Procedure
SSPORTS – Supervisor of Shipbuilding, Conversion and Repair, Portsmouth
UXO – unexploded ordnance

TABLE 1-3

DATA QUALITY OBJECTIVES FOR GEOTECHNICAL CONCERNS

STEP 1 Statement of Problem	STEP 2 Decisions	STEP 3 Input to the Decisions	STEP 4 Boundaries of the Study	STEP 5 Decision Rules	STEP 6 Limits on Decision Errors	STEP 7 Optimizing the Design
IR Site 2 contains a 77-acre, unlined landfill and a 33-acre wetland area. No maintenance has been performed. Waste depth is unknown. Waste delineation is not part of the geotechnical characterization. Contamination of soil or groundwater exceeding the TTLC hazardous levels is not anticipated. OEW/UXO could have been buried in the landfill. Engineered soil cover to be constructed over landfill, future reuse designated as a game refuge. Seismic and geotechnical evaluation is needed to determine the potential for slope failure into San Francisco Bay. Slope failure is a concern due to the potential release of waste into the bay.	What number of soil samples and tests are needed to characterize geotechnical parameters for the entire site? What are the existing data gaps that are needed to allow evaluation of seismic hazard exposure?	Historical document review will provide input for planning field testing program (number of CPTs, boreholes, locations, depths, sample types, sampling interval, sampling procedures, etc.) Field results (SPT blow counts, vane shear, and CPT test results) and laboratory tests will aid in evaluating the soil liquefaction potential and stability of perimeter dikes. Loading conditions will determine if UU, CD, or CU laboratory tests with pore water measurements will be performed. Data will include soil-strength characteristics and various loading conditions.	Roads/paved runways north and east of the site – San Francisco Bay to south and west (see Figure 4-2). Approximate area of investigation is described in Section 2.0 of the Work Plan (FWENC, 2002b). Tentative schedule for the fieldwork began December 2001. Project closeout is tentatively scheduled for 2003.	If the historic document review indicates that no data gaps exist, then FWENC will use available data. If not, then we shall proceed according to the Work Plan (FWENC, 2002b) and the results of historical document review. If critical slopes require additional stability and deformation analyses, then Phase 2 evaluation using Newmark-type deformation analysis methods will be used.	Due to judgmental sampling design, decision errors will not be established. The sampling plan criteria are based on a preliminary historical document review and past knowledge of the Bay Area geology and seismicity. Judgmental seismic interpretation can also occur in the field using the CPT and other seismic equipment and in analyzing field data (slope stability analyses).	Upland samples will be collected to a minimum of a 20-foot depth. Samples will be collected every 5 to 10 feet or at any change of formation based on the historical CPT results and field geologist/engineer observations. Similarly, the sample quantity for testing and laboratory testing program will be refined based on the past field test results. Locations of the analysis sections, initially selected based on the site topography (slope geometry), will be refined using the field and laboratory test data. Transect locations at 300-foot intervals were determined from past landfill field activity experience. Select interval locations will provide a continuous representation of the soil profile and in situ properties.

Notes:

CD – consolidated-drained
CPT – cone penetration test
CU – consolidated-undrained
FWENC – Foster Wheeler Environmental Corporation
IR – Installation Restoration
OEW – ordnance and explosives waste
SPT – standard penetration test
TTLC – Total Threshold Limit Concentration
UU – unconsolidated-undrained
UXO – unexploded ordnance

- **Locations of the analysis sections, initially selected based on the site topography (slope geometry), will be refined using the field and laboratory test data. Transect locations at 300-foot intervals were determined from the past landfill field activity experience. Select interval locations will provide a continuous representation of the soil profile and in situ properties** – additional cross sections were developed based on the field and laboratory test data.

- **Additional investigation** – an additional field investigation was conducted in the Additional Investigation Area between IR Sites 1 and 2 due to insufficient geological data that did not differentiate between various soil types or provide adequate information to extrapolate subsurface conditions.

Additional details regarding the geotechnical and seismic field investigation are included in Section 4.0 (Geotechnical and Seismic Evaluations).

1.5 REGULATORY FRAMEWORK

Environmental investigation and remediation of Alameda Point is being conducted under the Department of Defense (DoD) Installation Restoration Program (IRP). Details of the regulatory process and applicable or relevant and appropriate requirements (ARARs) were discussed in the Work Plan (FWENC, 2002b).

Regulated site activities performed at IR Site 2 included waste management and minimization of environmental impacts. Substantive ARARs were adhered to while conducting investigation activities and during excavation, demilitarization, and disposal of OEW materials.

1.5.1 Waste Management Activities

Several waste streams were generated during site activities at IR Site 2. Waste management activities included the management, storage, and eventual disposal or recycle of the waste streams.

OEW scrap (shrapnel, fins, and expended munitions) generated at the site were controlled and accounted for from discovery to disposal. Procedures for the accountability and disposition of OEW were presented in Appendix B, Standard Operating Procedure (SOP)-1 of the Work Plan (FWENC, 2002b). A total of 8,676 OEW scrap material items, which included 20 millimeter (mm) target practice/inert projectiles and an anti-tank/anti-personnel (AT/AP) inert training land mine were discovered at four locations within IR Site 2. OEW characterized as D003 reactive hazardous waste was not encountered during the course of activities at IR Site 2.

The AT/AP inert land mine was transferred to Navy personnel from the Explosive Ordnance Disposal (EOD) Mobile Unit 3, Southwest Detachment for return to the Navy Inert Ordnance Inventory. The 8,675 20mm target practice projectiles and casings were demilitarized in accordance with the DoD Defense Material Disposition Manual 4160.21-M-1, which specifies cutting each projectile in half. The demilitarized projectiles and casings were placed in a

55-gallon drum and disposed of as inert scrap metal at a Class III landfill (Forward Landfill, Manteca, California).

Soil cuttings and excavated materials from upland soil borings and test pit excavations were stockpiled adjacent to their point of origin. These materials, as well as drilling muds, were then used to backfill the boreholes and the test pits. Offshore soil cuttings and drilling muds naturally flowed back into the borings and produced no significant wastes. The designation of the site as an area of contamination under CERCLA allowed the placement (reconsolidation) of material generated during investigations within the same area of contamination without triggering land disposal restrictions or minimum technical requirements for a landfill.

Additional field investigation activities were conducted in an area between IR Sites 1 and 2. This area, which was not originally included in the SOW, is not recognized as an IR site under CERCLA. Therefore, the designation of the site as an area of contamination under CERCLA was not applicable and did not allow for the placement (reconsolidation) of material generated during investigations within the same area of contamination. Waste generated during field activities in the area between IR Sites 1 and 2 (drill cuttings and mud from rotary drilling) were placed in 55-gallon drums for waste characterization and subsequently disposed at the Kettleman Hills Landfill as non-hazardous investigative-derived waste under profile number EB 9426.

Prior to commencing waste storage activities, the FWENC Site Superintendent designated, in conjunction with the Environmental Compliance Manager (ECM), an area for the temporary staging and storage of drill cuttings and other anticipated miscellaneous waste streams. Secondary containment was provided for this temporary waste staging area.

OEW was stored in Magazine M353, located within a gated and locked compound. The magazine was protected from unauthorized access by a specialty security lock. The FWENC Senior UXO Supervisor (SUXOS) maintained control of the keys to both the magazine and the magazine compound.

An inventory of all waste containers was maintained for submittal and inspection by the Resident Officer in Charge of Construction (ROICC), as required. Containers of waste were inspected and logged weekly while the fieldwork was in progress. Inspections included evaluation for proper labeling, secure closure, the condition of each container, number of containers, and condition of the storage and secondary containment area.

Wastewater was not generated during the course of site activities. Dry decontamination of upland drilling equipment was performed by removing soil cuttings from auger heads and related equipment and placing within the location of the boreholes under the area of contamination designation. There were no waste fluids generated from heavy equipment activities at the site due to the short duration of time that the equipment was used. No equipment maintenance was conducted at the site that resulted in the generation of waste fluids.

1.5.2 Environmental Concerns and Mitigation

IR Site 2 consists of approximately 110 acres of coastal wetland including perennial ponds, a former landfill site, and a coastal margin composed of the perimeter dike and a riprap seawall. An interior margin lies outside the landfill and wetland to the north and east and is characterized as an upland area. The area outside of the berm was also used for waste disposal. Grasses and thistles are the dominant vegetation of the upland areas while seaside trefoil, Bermuda grass, and pickleweed inhabit the wetland area. IR Site 2 is currently used as a bird and wildlife sanctuary and is proposed for transfer to the USFWS for eventual use as a National Wildlife Refuge. Animals observed and known to inhabit IR Site 2 included black-tailed jackrabbit, feral cats, feral rabbits, ground squirrels, American peregrine falcon, red-tailed hawk, Canada geese, European starlings, western gulls, and red-winged blackbirds.

The wetland occupies a vegetated space that includes approximately 30 acres. The wetland consists of two ponds and adjacent areas that are inundated or saturated by surface or groundwater. No OEW or geotechnical characterization activities were conducted within the boundaries of the identified wetland areas during the course of the project.

Prior to the start of field activities at IR Site 2, all on-site personnel were briefed on the protection of natural resources including compliance with the intent of Section 404 of the Clean Water Act, which requires compensating for all wetland areas impacted by investigation or remediation activities. A qualified FWENC biologist performed the worker education briefing, emphasizing the need for minimizing impact on sensitive biological resources as well as methods for avoiding and minimizing potential impact on the species and communities of concern.

Field activities were not conducted during the California least tern nesting season designated as April through August. Therefore, no mitigation measures were necessary to reduce disturbance to the nesting populations adjacent to IR Site 2.

The following biological surveys were conducted to identify sensitive biological resources and other concerns related to field activities.

- On December 26, 2001, prior to the start of field activities at IR Site 2, a qualified FWENC biologist delineated the wetland boundaries by staking and placing pin flags along the wetland boundaries. This was performed to allow site personnel to visually identify wetland areas and avoid adverse impacts to the maximum extent possible.
- On February 26, 2002, a FWENC biologist inspected the 2.5-acre Possible OEW Burial Site. An evaluation was conducted to identify nests that could be harmed during the vegetation clearing process. No active nests were identified during the field survey, and vegetation clearing activities were performed following Navy review of the evaluation.
- Prior to the start of field activities related to the additional investigation between IR Sites 1 and 2, a FWENC biologist conducted a biological survey to determine the presence of California least tern and the effects of field exploration on the local

California least tern colony during the 2002 breeding season. No California least terns were discovered within 1,000 feet of the Additional Investigation Area.

No plant species found within the botanical ecosystem of IR Site 2 are state or federally listed sensitive species. All vegetation was mowed to a maximum height of 4 inches to facilitate the surface OEW clearance and intrusive investigation. Topsoil removed during intrusive excavation operations was replaced in the approximate stratigraphic depths from which it was removed. Although natural resource mitigation measures were not implemented during the course of the activities at IR Site 2, monitoring personnel conducted periodic site surveys during the initial ground disturbing activities to oversee and record activities resulting in terrestrial disturbance.

1.5.3 Agency Notifications

The Alameda Point ECM/Caretaker and the following agencies were notified at least 60 days prior to the start of operations on IR Site 2. Final Work Plans, which included a Primavera project schedule detailing start date, individual task duration, and demobilization date, were submitted to the Alameda Point ECM/Caretaker and each of the following agencies:

- EPA
- Regional Water Quality Control Board (RWQCB)
- Department of Toxic Substances Control (DTSC)
- The City of Alameda

Prior to conducting the offshore drilling at IR Site 2, the Coast Guard was notified in order to file a Notice of Mariners and Waiver of Anchorage that documented the offshore drilling activities and radio frequency of the drilling vessel for communication with channel traffic.

1.5.4 Spills and Releases Control

Precautions were taken to prevent hazardous material spills. Daily inspections by site personnel of equipment, structure(s), and containers were conducted. In addition, personnel using hazardous materials inspected containers before and after use. In the event of a spill/release, the Site Superintendent was required to notify the Navy, and a spill response effort would be conducted in accordance with the *Final Base-Wide Health and Safety Plan* (FWENC, 2001) and federal, state, and local regulations, and in accordance with Navy policies and procedures. There were no spills or releases of hazardous materials during investigation or excavation activities conducted at IR Site 2.

1.5.5 Applicable Regulations and Criteria for Geotechnical and Seismic Design

No specific guidelines or regulations have been provided for seismic stability evaluation of the IR Site 2 landfill. Therefore, the existing regulations for seismic design of landfills that include guidelines for seismic evaluation and design of landfill closure systems were used as guidelines for IR Site 2 stability evaluations.

1.5.5.1 State and Federal Regulations

The siting, design, permitting, and construction of new solid waste disposal facilities or expansion and closure of existing facilities must meet the requirements of Title 27 California Code of Regulations (CCR), implemented by the California Integrated Waste Management Board (CIWMB) and State Water Resources Control Board (SWRCB). The Code of Federal Regulations (CFR), Section 40, Part 258 (commonly known as Subtitle D), applicable to all solid waste landfills in the country, were adopted by the CIWMB by amending Title 14 CCR regulations (now Title 27 CCR) and by SWRCB by adopting Resolution No. 93-62, Policy for Regulation of Discharges of Municipal Solid Waste. Accordingly, SWRCB issued a master version of the blanket Waste Discharge Requirement (WDR) that incorporates both CFR Subtitle D and Title 27 CCR (formerly Title 23 CCR, Chapter 15) regulations. Each RWQCB has flexibility to implement slightly different versions of the blanket WDR.

Title 22 CCR addresses seismic and precipitation design standards for hazardous waste landfills (Class I). Title 22 CCR has not been determined to be applicable because the landfill has not been classified as hazardous (Class I). However, it is still a relevant and appropriate requirement based on the nature of the wastes historically disposed of in the unit. Prior historic information gathered for the *Initial Assessment Study of Naval Air Station, Alameda*, (E&E, 1983) states that the site contains a mixture of municipal solid waste, waste chemical drums (contents unknown), solvents, oily waste and sludge, paint waste, plating wastes, industrial strippers and cleaners, acids, mercury, PCB-containing liquids, batteries, low-level radiological waste from radium dials and dial painting, scrap metal, inert ordnance, asbestos, several pesticides (solid and liquid), tear gas agent, biological waste from the Oak Knoll Naval Hospital, creosote, dredge spoils, and waste medicines and reagents. In addition, a TCRA was performed at the 2.5-acre (approximate) Possible OEW Burial Site located in the southern part of the landfill. During the TCRA, 8,675 20mm target practice projectiles were uncovered. The projectiles did not contain any explosives or energetics. The heterogeneity of contaminant distribution and concentrations typically associated with landfills makes accurate characterization of landfill refuse impractical and virtually impossible. Also, no invasive work was conducted as part of the geotechnical investigation to either characterize or delineate the area of refuse within the IR Site 2 disposal area. Therefore, while a formal determination cannot be made regarding hazard classification of IR Site 2, this is immaterial since the Navy's position, in accordance with EPA policy, is to apply relevant and appropriate requirements to the same degree as if they are applicable.

The following paragraphs provide detailed discussions of the existing applicable regulations for seismic stability evaluation of landfills.

Requirements for the stability analyses of Class III landfills (landfills for non-hazardous solid waste) are contained in Sections 20370 (f) and Section 21750 (f) (5) of Title 27 CCR and CFR, Section 40, Part 258. Title 27 CCR requires "Class III waste management units to be designed to withstand the Maximum Probable Earthquake (MPE) without damage to the foundation or to the structures which control leachate, surface drainage, erosion, or gas."

California Divisions of Mines and Geology (CDMG) Note No. 43 defines the MPE as "the maximum earthquake that is likely to occur during a 100-year interval ... the postulated magnitude of the MPE is superseded by any more powerful seismic event that has occurred within historic time in the area." It is to be regarded as a probable occurrence, not as an assured event that will occur at a specific time. This definition of MPE has normally been interpreted as a seismic event having an average return period of 100 years.

For Class II landfills (waste management units for designated waste) and Class I landfills (hazardous waste landfills), Title 22 and 27 CCR require consideration of the Maximum Credible Earthquake (MCE) for seismic stability design. MCE is defined by CDMG as "the maximum earthquake that appears capable of occurring under presently known tectonic framework." By definition, for the same set of faults, the MCE generally will result in a larger earthquake compared to the MPE. The MCE is evaluated for faults determined to produce potentially damaging ground motions at the site. The analyses will include effects of both near-field and far-field/intermediate-field seismic events to ensure that higher intensity, shorter duration and lower intensity, longer duration earthquake ground motions are considered. The following provides more details on stability evaluation of Class II and III and landfills, as described in Title 27 CCR.

For static stability, only qualitative requirements are indicated in the cited regulations. The current state of practice in California for static design is to require a minimum factor of safety of 1.5 for all final waste slopes. Section 21750 (f) (5) of Title 27 CCR (Seismic Design) calls for:

> A stability analysis, including a determination of the expected peak ground acceleration at the Unit associated with the maximum credible earthquake (for Class II waste management units) or the maximum probable earthquake (for Class III landfills)...The methodology used in the stability analysis shall consider regional and local seismic conditions and faulting...
>
> (A) The stability analysis shall ensure the integrity of the Unit, including its foundation, final slopes, and containment systems under both static and dynamic conditions throughout the Unit's life, closure period, and post-closure maintenance period....
>
> (C) The stability analysis shall be prepared by a registered civil engineer or certified engineering geologist. Except as otherwise provided in ¶(f)(5)(D), the report must indicate a factor of safety for the critical slope of at least 1.5 under dynamic conditions....

(D) In lieu of achieving a factor of safety of 1.5 under dynamic conditions, pursuant to ¶(f)(5)(C), the discharger can utilize a more rigorous analytical method that provides a quantified estimate of the magnitude of movement. In this case, the report shall demonstrate that this amount of movement can be accommodated without jeopardizing the integrity of the Unit's foundation or the structures, which control leachate, surface drainage, erosion, or gas.

In addition to the seismic stability requirements of Title 27 CCR described above, Section 66264.25 (b) of Title 22 CCR specifies seismic design requirements for hazardous waste landfills as follows:

> "The following shall be designed, constructed and maintained to withstand the maximum credible earthquake without the level of public health and environment protection afforded by the original design being decreased:
>
> (1) all surface impoundments, waste piles, landfills and land treatment facilities subject to this chapter; and
>
> (2) all covers and cover systems required by this chapter and all containment and control features which will remain after closure at permanent hazardous waste disposal areas."

Design of IR Site 2 landfill closure will follow the requirements of Title 27 CCR, which provide guidelines for Class II (designated waste) and Class III (non-hazardous solid waste) landfills. Title 22 CCR addresses seismic and precipitation design standards for hazardous waste landfills (Class I). As discussed previously, Title 22 CCR has not been determined to be an applicable requirement since no formal classification for the landfills at IR Site 2 has been established. However, it is still a relevant and appropriate requirement due to the nature of the wastes historically placed into the landfill. Therefore, the proposed remedy must meet both the Title 27 and the Title 22 standards. In order to satisfy the requirements of both Title 22 CCR and Title 27 CCR pertaining to seismic design, the more conservative maximum credible earthquake scenario was used as the basis for seismic design.

Title 27 CCR only refers to evaluation of dynamic stability (stability during earthquake shaking) when landfill slopes are subjected to seismic loading. In addition to Title 27 CCR requirements, post-earthquake static slope stability evaluations are also required in accordance with the United States Army Corps of Engineers (USACE) Manual EM 1110-2-1913 guidelines (2000a) for seismic stability evaluation of levees. For slopes comprised of or founded on materials in which their strength properties change considerably when subjected to strong ground shaking (for example, liquefiable soils), post-earthquake static stability analyses using residual strength properties are performed to evaluate the potential for slope failure after earthquake shaking terminates. For post-earthquake stability conditions, according to the USACE Manual EM 1110-2-1913 (2000a), the minimum acceptable factor of safety is 1.0.

1.5.5.2 Design Basis

No formal classification has been established for landfills at IR Site 2 as of this time. However, the RWQCB has indicated that IR Site 2 should be designated as a Class II waste management unit (landfills for designated waste). Title 27 CCR requires that Class II landfills be designed for the MCE. Title 22 CCR also requires that Class I landfills be designed for the MCE. For Class III landfills (landfills for non-hazardous solid waste), Title 27 CCR requires the use of the MPE. In general, the MCE results in a larger predicted earthquake than the MPE. In order to satisfy the ARARs of both Title 22 CCR and Title 27 CCR, it was decided to use the MCE for seismic stability evaluations of IR Site 2 and the Additional Investigation Area between IR Sites 1 and 2.

For seismic stability, a pseudo-static factor of safety greater than 1.0 is considered acceptable when designing for the PHGA. This indicates that no seismically induced displacements will occur even when the PHGA is encountered. When the pseudo-static factor of safety is less than 1.0, the slope yields, and seismically induced permanent displacements will occur. Current engineering practice is to calculate the seismically induced displacements of the landfill slopes using a Newmark (Newmark, 1965)-equivalent method (Seed and Bonaparte, 1992). For lined landfills, the allowable seismically induced slope displacements along liners are commonly set to a maximum of 6 inches to 1 foot.

For cover systems, there is no maximum deformation specified. Regulations simply indicate that the cover system must "withstand earthquake loading." However, because cover repairs can be made more easily than liner repairs, current practice is to allow a greater level of deformation and, although 1 foot of deformation has been used in practice, that is to be evaluated on a case-by-case basis.

For IR Site 2, since it is an unlined landfill and will eventually be transferred for end use as a wildlife refuge, larger permanent seismically induced slope displacements on the order of several feet may be allowed. Selection of a more precise value for the allowable seismic design displacement depends on the following factors:

(1) Width of the buffer zone between the waste limit and the shoreline along San Francisco Bay on the west side of the site.

(2) The nature of the remediation measure(s) that may be used to limit the seismic displacements of the landfill perimeter slopes. For example, if stone column lines are used to confine the landfill and enhance seismic stability, the width of the stone column wall will dictate the allowable seismic displacements of the stabilized slopes.

The allowable seismic slope displacements will be evaluated as part of the Geotechnical Feasibility Study of IR Site 2.

1.5.6 Applicable Regulations and Criteria for OEW Management

DoD and Navy Regulations

DoD and Navy regulations focus primarily on the management of OEW as a potentially reactive (D003) hazardous waste. Because the remediation project is being conducted on a BRAC site, DoD and Navy publications govern the handling, storage, transportation, clearance, and disposal requirements for UXO. They broadly apply and are applicable to all UXO activities on federal property as follows:

- Naval Sea Systems Command (NAVSEA). 2001. *Ammunition and Explosives Ashore Safety Regulations for Handling, Storing, Production, Renovation and Shipping*. U. S. Navy Manual (NAVSEA) OP-5. Revision 7. January.
- DoD. 1996. *DoD Contractor's Safety Manual for Ammunition and Explosives.* DoD Instruction 4145.26M. April.
- DoD. 1999. *DoD Ammunition and Explosives Safety Standards, DoD Explosive Safety Board*. DoD 6055.9-STD. July.
- USACE. 2000b. *Final Management Principles for Implementing Response Actions at Closed, Transferring and Transferred Ranges Action Memorandum*. December.

Other Federal/California ARARs/To Be Considered (TBC) Requirements

Other federal agencies' requirements that are potential ARARs include:

- **Military Munitions Rule (MMR)** (Title 40 CFR, Parts 260 through 270). Requirements for waste military munitions (WMM), transportation, treatment, and disposal of WMM and response to WMM/explosives emergencies.

2.0 WETLAND ASSESSMENT AND SITE SURVEYS

This section describes the wetland assessment and site survey activities associated with the upland and offshore investigation at Installation Restoration (IR) Site 2. The wetland assessments involved conducting a biological survey, which consisted of evaluating the impact of site activities on the wetland areas and various animal species inhabiting IR Site 2 and the Additional Investigation Area between IR Sites 1 and 2. The civil survey activities include performance of a bathymetric and topographic survey of the site. In addition, grid networks and location points were established for the ordnance and explosives waste (OEW) characterization and geotechnical field investigation respectively. The survey work was completed in accordance with the *Final Focused Remedial Investigation Work Plan* (Work Plan) [Foster Wheeler Environmental Corporation (FWENC), 2002b].

2.1 WETLAND ASSESSMENT

In November 2001, FWENC biologists conducted a wetland assessment to determine the potential impacts on wetland and water resources from the OEW characterization, Time-Critical Removal Action (TCRA), and geotechnical and seismic evaluations at IR Site 2. This biological study was performed to identify the location and boundaries of all jurisdictional wetland and waters within the proposed work area subject to jurisdiction by the United States Army Corps of Engineers (USACE) under Section 404 (b)(1) of the Clean Water Act.

On February 26, 2002, FWENC biologists evaluated the proposed project excavation area for the presence of nesting birds protected under federal and California state laws, Migratory Bird Treaty Act (MBTA) and Fish and Game Code, Sections 3503 and 3503.5. The evaluation was conducted to prevent the taking of nesting birds during the vegetation clearing process. In addition, on June 16, 2002, FWENC biologists evaluated the Possible OEW Burial Site and Additional Investigation Area (between IR Sites 1 and 2) for the presence of nesting California least terns (*Sterna antillarum*). The evaluation was conducted to determine if field exploration activities could potentially affect the local California least tern colony during the 2002 breeding season.

The results of the wetland and avian surveys are presented in the following discussion. Potential impact to the site is discussed at the end of the section.

Wetland Survey

Wetland habitats associated with permanent water sources, as well as intermittent drainage channels, provide food, water, migration and dispersal corridors, nesting and breeding habitat, and contain habitat that is distinct from the adjacent uplands for a variety of wildlife species. Numerous amphibian, reptile, bird, and mammal species are residents or visitors in wetland habitats due to the vegetation's structural diversity. Wetland habitats are essential breeding, rearing, and foraging grounds for many species of wildlife. Wetlands also perform important flood protection and pollution controls.

A wetland delineation evaluating vegetation, soil, and hydrology of potentially jurisdictional areas within the IR Site 2 work area was conducted in accordance with the procedures of the USACE *Wetlands Delineation Manual* (USACE, 1987). Potential jurisdictional wetlands found within the project study area are listed in Table 2-1 and shown in Figure 1-2.

Wetland WE1: Salt Marsh – Estuarine Intertidal Persistent Emergent Wetland Community

This wetland occupies a vegetated space along the western coastline of Alameda Point. The wetland is bounded by a landfill to the north and east and is adjacent to San Francisco Bay on the south and west. It consists of approximately 29.3 acres of salt marsh wetland habitat. Due to a prevalence of obligate and facultative hydrophytic vegetation, abrupt wetland boundary, and the direct observation of inundated and saturated soil, a hydric soil condition was inferred (USACE, 1987). Hydrology from tidal fluctuations, upland runoff, precipitation, and a high groundwater table support the hydrophytic vegetation present at this site. Standing water and saturated soils were observed at the surface. The wetland contains two perennial ponds. The northern pond is connected to the bay by a culvert, and the southern pond was created by the removal of dredged materials for use as landfill cover. Salt water has filled the northern pond and fresh water has filled the southern pond. The dominant vegetation consists of salt marsh pickleweed (*Salicornia virginica*), obligate wetland vegetation (OBL), and Bermuda grass (*Cynodon dactylon*) [faculative vegetation (FAC)]. All of the dominant plant species observed were obligate or facultative in nature.

Wetlands WE2 and WE3: Seasonal Wetland Communities

These wetlands occupy a vegetated space approximately 1,600 feet east of the western coastline of Alameda Point. The wetlands are adjacent to San Francisco Bay on the south and west. Wetland WE2 is approximately 0.2 acres, and wetland WE3 is approximately 0.03 acres of seasonal wetland habitat at the northeastern edge of the study area. Soils were identified by digging a soil pit to a depth of 12 inches within a topographic low of a basin positioned to the east of San Francisco Bay. The soil is a sandy loam with a matrix color 7.5YR 3/1 with mottling color of 2.5YR 4/8. Hydric soils were determined to be present due to low-chroma color of the substrate and high organic content in the surface layer. Hydrology for this wetland is provided from the low groundwater table resulting from the close proximity of the wetlands to the San Francisco Bay. Hydrology from upland runoff, precipitation, and surface flows also support the hydrophytic vegetation present at this site. The depth to water is 12 inches, and saturated soils were observed in the first inch of the soil pit. Additionally, sediment deposition and drainage patterns were observed in these wetland features. The dominant vegetation consists of Bermuda

TABLE 2-1

POTENTIAL JURISDICTIONAL WETLANDS WITHIN THE STUDY AREA

Wetland I.D. Number	USGS Quad Name	Acreage of Impact	Acreage of Wetland	Classification	Vegetation
WE1	Oakland West	0	29.3	Salt Marsh – Estuarine Intertidal Persistent Emergent Wetland	Bermuda grass, salt marsh pickleweed
WE2	Oakland West	0	0.2	Seasonal Wetland	Bermuda grass, curly dock
WE3	Oakland West	0	0.03	Seasonal Wetland	Bermuda grass, curly dock

Notes:

USGS – United States Geological Survey

grass (*Cynodon dactylon*) (FAC), and curly dock (*Rumex crispus*) [facultative wetland vegetation (FACW)]. All of the dominant plant species observed were obligate or facultative in nature.

Avian Inspections

The project activities at IR Site 2 incorporated a number of measures to minimize adverse impact to bird species listed as Endangered, Threatened, or Candidate species under federal and California state laws, as well as to certain other species which receive protection under the California Department of Fish and Game (CDFG) codes and the MBTA. In February 2002, FWENC evaluated the proposed project excavation area for the presence of nesting birds. The evaluation was conducted to prevent the taking of nesting birds during the vegetation clearing process. FWENC biologists examined all specimens proposed for removal immediately prior to the onset of vegetation clearing. Potential habitat that was identified include several semi-mature Sydney Golden Wattle *Acacia longifolia* and one immature Spruce (*Picea sp).* The tree specimens contained several small passerine nests. All of the nests that appeared to be from last season were unoccupied and showed no physical signs of recent activity (no whitewash, feathers, or other signs were discovered). The nests were void of fresh sign, and cobwebs and spider webs were observed.

In June 2002, FWENC evaluated the Possible OEW Burial Site and Additional Investigation Area for the presence of nesting California least terns (*Sterna antillarum*). The evaluation was conducted to determine if field exploration activities and continued maintenance of the excavation cover (at the Possible OEW Burial Site) could potentially affect the local California least tern colony during the 2002 breeding season. FWENC biologists examined the field exploration activities, the entire covered excavation area, and adjacent lands for the presence of California least terns. No nesting or foraging California least terns were discovered within 1,000 feet of the field exploration activities, or the excavation area.

Project Impacts

The project field activities conducted did not result in the permanent loss of any jurisdictional wetland. More specifically, no permanent above-grade fills were constructed within any jurisdictional wetland. No investigation or characterization activities were performed within the boundaries of any wetland areas. Areas with the potential to provide habitat to species of concern were identified prior to activities, and staked for avoidance where necessary. Site selection for project staging areas, where hazardous materials and hazardous wastes may be present, were considered and wetlands were avoided. No active nests protected under federal and California state laws, MBTA, and CDFG Code Sections 3503 or 3503.5 were identified during the field evaluations. No nesting or foraging California least terns were discovered within 1,000 feet of the field exploration activities or the excavation area.

The study area is currently used as a bird and wildlife sanctuary and is proposed for transfer to the United States Fish and Wildlife Service (USFWS) for use as a National Wildlife Refuge. Wildlife species that are federally listed as endangered or threatened could potentially occur on IR Site 2, based on their presence at similar areas in Alameda County. These species include the winter-run chinook salmon, tidewater goby, California brown pelican, California clapper rail, western snowy plover, California least tern, American peregrine falcon, Steller sea lion, and salt marsh harvest mouse. None of these species are known to currently inhabit IR Site 2 (nesting California least tern colony is over 1,000 feet away), and they will not be prohibited from IR Site 2 in the future as a result of remedial activities that took place at the site.

2.2 SITE SURVEY

Kister, Savio, and Rei, Inc. (KSR), licensed land surveyors in the state of California, performed site surveys and data interpretation for IR Site 2, from January 2002 through April 2002. KSR performed these surveys to establish control for the site, establish a grid for the OEW sweep and provide design and as-built locations for the cone penetration test (CPT), soil boring, and test pit locations used for the geotechnical characterization.

2.2.1 Surveying and Site Control

The survey control for the site was based on a monument located at the northwest corner of Main Street and Atlantic Avenue in the city of Alameda. The location of the monument was provided by the Navy and described by the National Geodetic Survey (NGS) as "Main/Atl". The NGS defines and manages the National Spatial Reference System (NSRS) – the framework for latitude, longitude, height, scale, gravity, orientation, and shoreline throughout the United States.

The site coordinates are currently based on the California Coordinate System (CCS) Zone III, North American Datum (NAD) of 1927. The NAD27 value for "Main/Atl" was derived from the NADCON conversion of the published NAD83 coordinates. The coordinates for the control point at "Main/Atl" based on NAD27 are provided as follows:

- Northing - 471,068.97
- Easting - 1,482,604.56

The site elevations are based on the National Geodetic Vertical Datum (NGVD) of 1929. The published elevation of the "Main/Atl" is provided as follows:

- Elevation - 6.69 feet

Figure 2-1 accurately depicts the location of the investigation points around the site, the limits of the IR Site 2 area, and the current shoreline. The OEW grid system is described in Section 3.1.

Land survey data are included in Appendix D.

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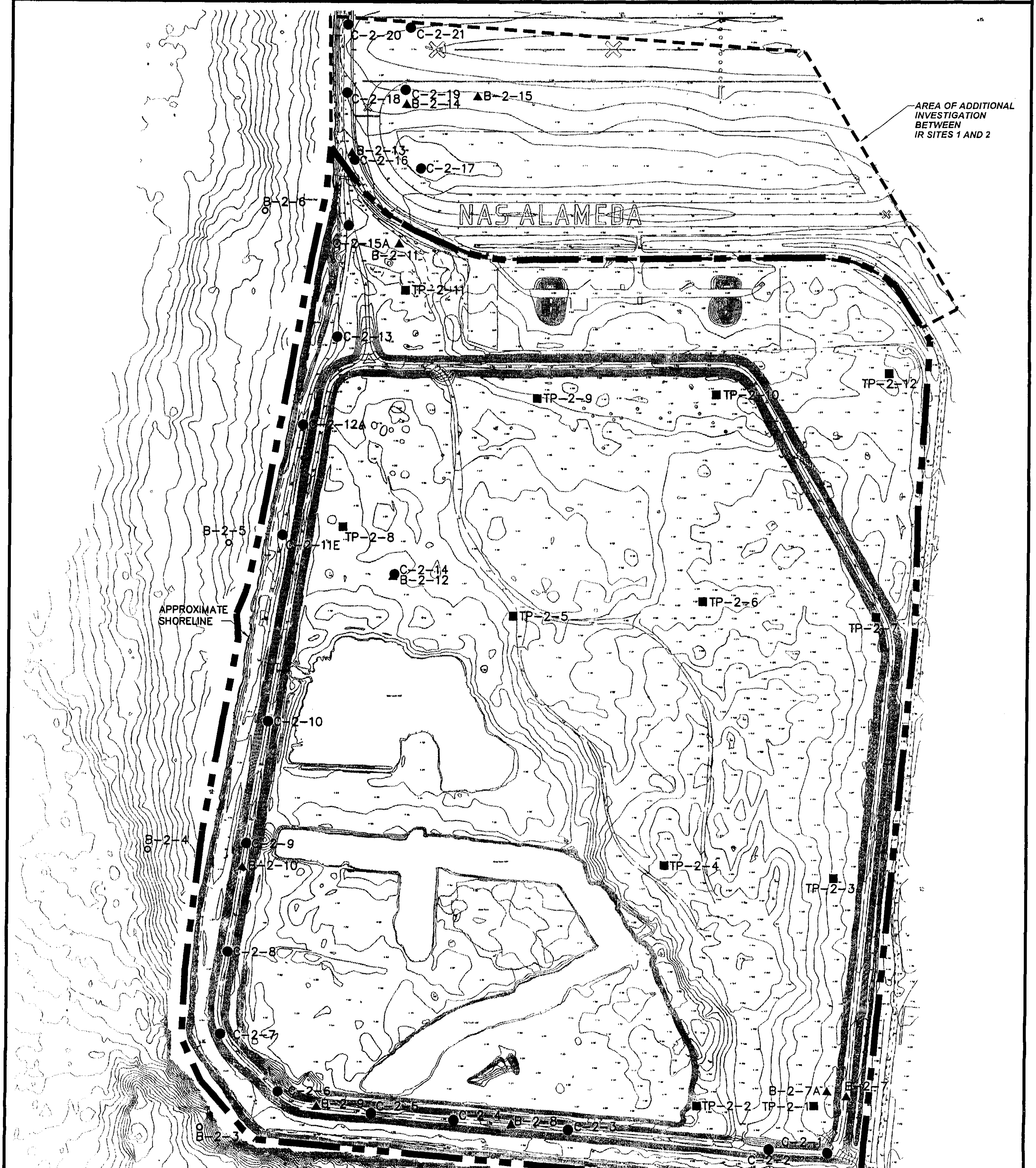

NOTE:

1. ALL C,B AND TP DESIGNATIONS ARE AS-BUILT LOCATIONS FOR SUBSURFACE EXPLORATION. AN "A" DESIGNATION REPRESENTS AN AS-BUILT LOCATION THAT WAS ADJUSTED FROM THE DESIGN LOCATION DUE TO A SURFACE OR SUBSURFACE OBSTRUCTION ENCOUNTERED DURING FIELD ACTIVITIES.

LEGEND

● C	CPT SOUNDINGS
▲ B	UPLAND BORINGS
■ TP	TEST PITS
○ B	OFFSHORE BORINGS
▬ ▬ ▬	SITE BOUNDARY

N

300 150 0 300

SCALE IN FEET

Figure 2-1
IR SITE 2 COMBINED TOPOGRAPHY/BATHYMETRY MAP WITH FIELD EXPLORATION LOCATIONS

Southwest Division
Naval Facilities Engineering Command

TETRA TECH FW, INC.

2.2.2 Topographic Survey

HJW GeoSpatial, Inc., prepared the topographic map using aerial photographic cover panels and computer-assisted, photogrammetric methods. Photographs used for the map construction were of IR Sites 1 and 2 on Alameda Point and were collected March 2002. The topographic map was produced in April 2002. The topographic map contour intervals are 1 foot and are based on the CCS Zone III, NAD27. Map elevations are based on NGVD29.

The topographic map is presented in Appendix E as an oversized drawing. The map is used as the base map for select figures presented in this report.

2.2.3 Bathymetric Survey

EcoSystems Management Associates, Inc. (EcoSystems) was subcontracted to perform a USACE Class 1 hydrographic survey. On January 4, 5, 6, 7, 29, and 30, 2002, EcoSystems surveyed the IR Sites 1 and 2 offshore areas that were accessible by survey vessel. This survey was conducted to a distance of approximately 500 feet offshore. Survey lines were established normal to the general shoreline orientation at 50-foot intervals. Tie lines were set up to intersect the survey lines at an approximate 100-foot spacing from the shoreline to the offshore limit of the survey area.

HydroPro hydrographic surveying software was used for navigation and to record real-time position, depth, and tide correction. A 27-foot survey vessel equipped with a side-mounted narrow beam (3o) 200 kilohertz (kHz) transducer and Odec Bathy 500 multi-frequency fathometer was used to conduct the survey. A Leica MX300 digital global positioning system (DGPS) and Sokkia Starlink (Model 1071) were used during the survey to determine the real-time position. The DGPS position was differentially corrected with a U.S. Coast Guard DGPS broadcast correction. A tide gauge (Microtide manufactured by Coastal Leasing) was installed on the "T" pier in the Oakland Inner Harbor entrance channel to record water surface elevation for the duration of the surveys. Vertical control for the tide gauge was based on Bench Mark No. 500 (Figure 2-2). This data was used to correct survey data to the vertical project datum.

The bathymetric survey data was post-processed, combined, and provided to FWENC as an ASCII file and an AutoCAD 2000 drawing file on February 8, 2002. The bathymetric data has been incorporated into the site topographic map (see Figure 2-1). A report and an oversized bathymetry survey drawing by EcoSystems Management Associates, Inc. (EcoSystems) are included in Appendix D.

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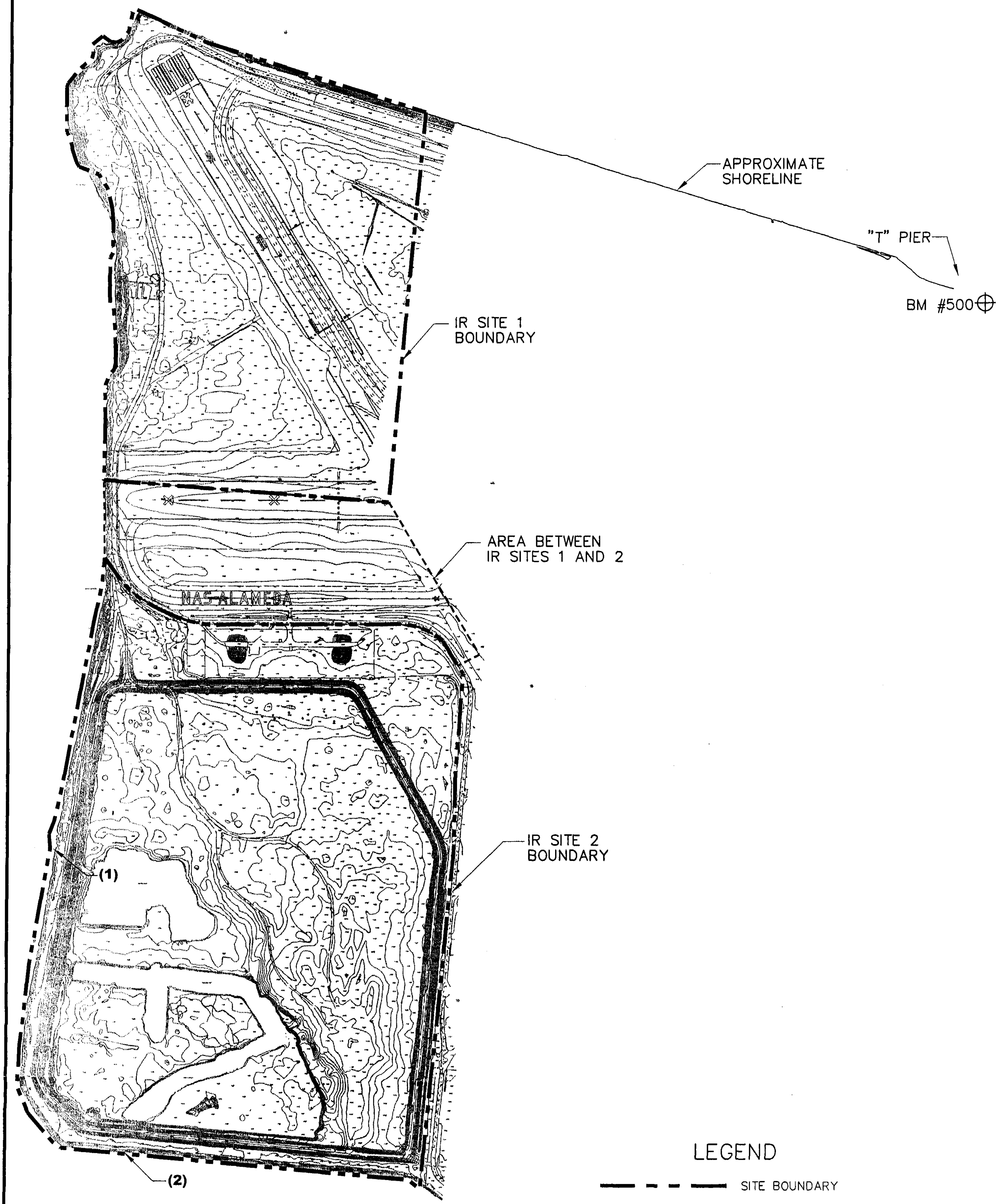

LEGEND

— — —	SITE BOUNDARY
⊕	BENCHMARK (BM 500)
(1)	DESIGN ACCELERATION (POINT NO. 1)
(2)	DESIGN ACCELERATION (POINT NO. 2)

N

500 250 0 500
SCALE IN FEET

REFRENCE:
HJW-GeoSpatial, Inc., Upland topography
NAD27, NGVD29 - CCS Zone III.

Figure 2-2
IR SITE 2 TOPOGRAPHIC MAP

Southwest Division
Naval Facilities Engineering Command

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3.0 ORDNANCE AND EXPLOSIVES WASTE CHARACTERIZATION

This section presents an overview of the ordnance and explosives waste (OEW) characterization performed at Installation Restoration (IR) Site 2. The discussion includes a summary of the quality control (QC) procedures, characterization results, and disposition of recovered OEW.

Prior to conducting any field activities, a visual reconnaissance of access roads, staging areas, and support zones was performed to remove potentially hazardous OEW, metal, and other debris from the ground surface that could have interfered with ordnance detection equipment. Vegetation was cut to a height of no more than 4 inches in the upland areas to facilitate the surface OEW characterization of the entire site and to provide access for the Time-Critical Removal Action (TCRA), soil sampling activities, and test pit explorations. Unexploded ordnance (UXO) technicians proceeded ahead of the mowing equipment to prevent contact with OEW. The vegetation was low-growth and the cuttings were left on the site. A few small trees were uprooted in the area where the TCRA occurred. No work was conducted on land within established wetland boundaries.

Several treatment alternatives for the disposal of encountered OEW were considered and included contained detonation, off-site shipment to an approved disposal facility, transfer of the material to the military, and on-site incineration. Open detonation was not considered a viable method for treating OEW that could be certified safe to ship. Other alternatives were considered which were determined to be safer and less damaging to the environment. Procedures for open detonation were developed to address fuzed and fired OEW that are unsafe to ship and presented a threat to human health or the environment. If a situation had occurred that required open burn/open detonation, an Emergency Removal Action would have been performed that included engineering controls to contain/control the open detonation. Transfer of OEW to the military and on-site incineration were considered prohibitive. Therefore, off-site shipment was selected as the preferred option.

3.1 SURFACE CHARACTERIZATION

The site grid used for UXO characterization was surveyed by Kister, Savio, and Rei, Inc. (KSR). The grid was based on horizontal and vertical control (benchmark) located at Main and Atlantic streets in Alameda, California. A 200-foot by 200-foot grid was installed, as shown in Figure 3-1, over the upland areas of IR Site 2. The vertical datum was based on the National Geodetic Vertical Datum (NGVD) of 1929 and horizontal survey datum to the California Coordinate System (CCS) Zone III, North American Datum (NAD) of 1927. The NAD27 values for the Main/Atlantic monument were derived from a conversion of published NAD83 coordinates. The grids were superimposed on a computer-assisted drawing (CAD) map of Alameda Point. After the grid network was established, the UXO team conducted a systematic grid-by-grid sweep of the site. The UXO team formed a line abreast spaced in a manner that

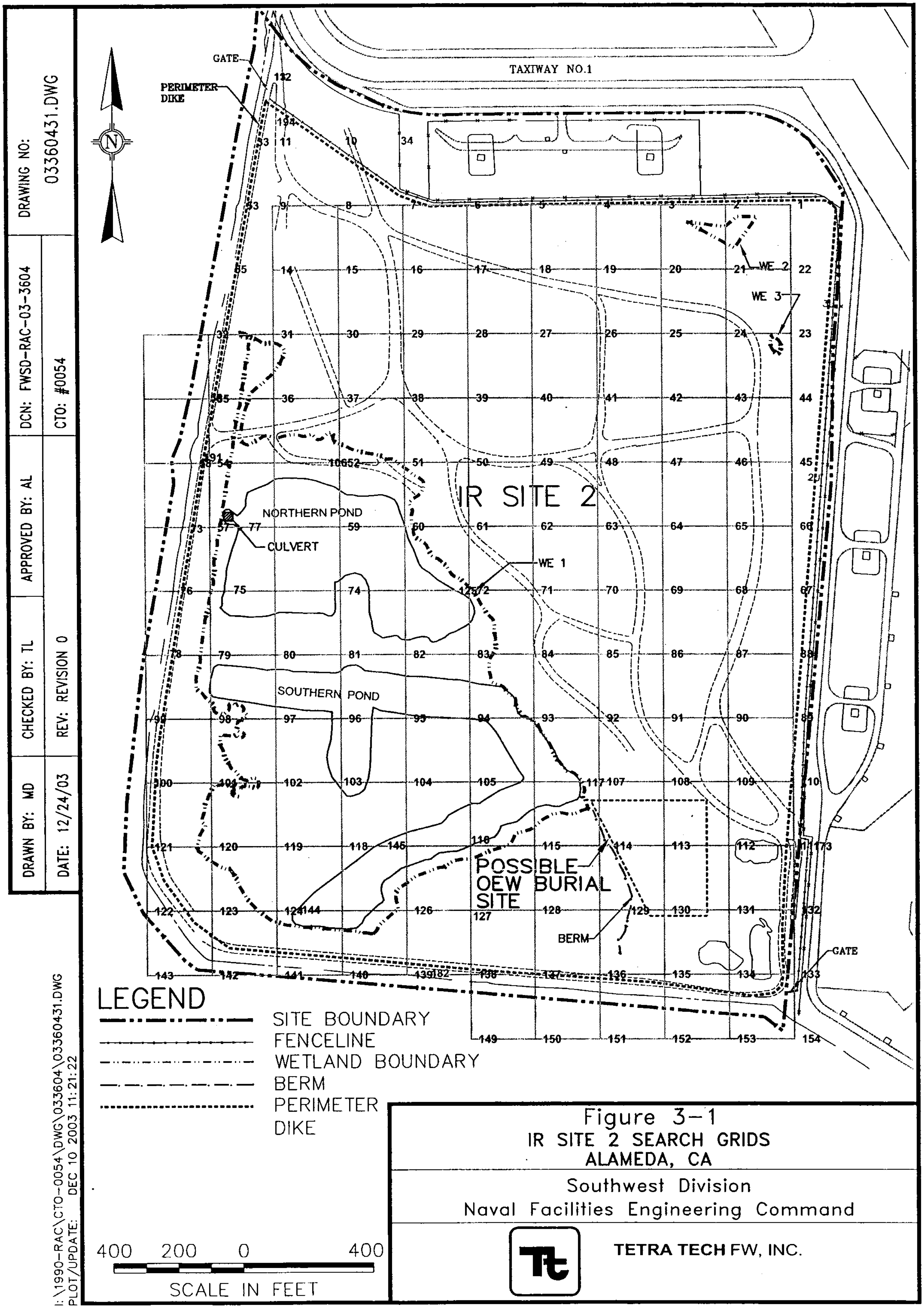


Figure 3-1
IR SITE 2 SEARCH GRIDS
ALAMEDA, CA

Southwest Division
Naval Facilities Engineering Command

TETRA TECH FW, INC.

permitted a slight visual overlap of individual lanes. The team member on one end of the line acted as the guide and navigated a straight path between grid boundaries using the installed grid stakes as initial guideposts. A bright orange traffic cone was placed on the ground adjacent to the person on the end of the line opposite the guide. The UXO team maintained alignment and spacing with the guide as the sweep proceeded. When the team reached the opposite end of the grid, the line stopped, and another traffic cone was placed on the ground marking the outside boundary and stopping point for that particular sweep. The traffic cones were positioned in a manner that allowed a slight overlap of the sweep lanes and then became the guideposts for the next sweep. Each team member swept the probe of a Schonstedt GA-52 CX ordnance locator in small arcs in front of them as they proceeded (this technique focused the vision on the ground in front of them and provided an audible backup). This process was followed until the grid was cleared and then repeated in every grid until the remaining upland areas of IR Site 2 were swept.

The Schonstedt GA-52 CX was used to conduct the surface OEW characterization rather than the MK 26, which is the standard issue magnetometer used by Explosive Ordnance Disposal (EOD) Units, because Schonstedt GA-52 CX is more versatile and just as effective for conducting surface and near-surface charaterization. The Foerster Ferex® 4.021/MK 26 is a versatile, supersensitive search instrument with a sole purpose of locating ferromagnetic items buried in the ground or underwater at depths of up to 6 meters. During operations, the sensor probe is held stationary and is moved in parallel lines over the area to be searched in lanes approximately 1 meter apart. It weighs nearly 14 pounds and requires two hands to operate. The Schonstedt GA-52 CX is over 10 pounds lighter than the MK 26, requires only one hand to operate and can detect large, subterranean ferromagnetic items at depths approaching 3 meters. It is swept side-to-side in front of operators as they proceed down search lanes during a surface characterization of an area. This technique helps personnel to concentrate on the ground in front of the probe as they walk. The Schonstedt GA-52 CX was also used for OEW avoidance procedures during test pit excavations. The Shonstedt MG 220 magnetic locator was used for OEW avoidance procedures in boreholes.

The location of OEW encountered during the sweep was referenced by an abscissa/ordinate intersection point of appropriate alphanumeric label of the grid's placement within the coordinate system. Locations of items found during the characterization were identified by northing and easting distances from the southwest grid stake and plotted on the CAD site map. Any suspected or known OEW encountered was clearly marked and its position annotated on the site map. The Senior UXO Supervisor (SUXOS) evaluated all encountered OEW and determined if the characterization work could safely proceed. The UXO team identified areas with tape or flags when OEW was encountered, and only essential UXO team members were allowed into the zone until the SUXOS determined that no hazard existed.

The data will be uploaded into the Geographic Information System (GIS) for Alameda Point. Digital photographs were taken of items found during the characterization and excavation activities. The photographs were recorded in the project photograph log, which is part of the project files.

3.2 EXCLUSION ZONE MARKING AND CONTROL

Exclusion zones (EZs) are areas where contamination (hazards) are known or likely to be present, or areas that, because of activity, have the potential to cause harm to personnel. The EZ for high explosives is determined by the amount of explosives an OEW item contains and how it is configured. Based on the results of earlier radiological surveys and a previous Emergency Removal Action [Supervisor of Shipbuilding, Conversion and Repair, Portsmouth (SSPORTS), 1998], the 20 millimeter (mm) high-explosive incendiary (HEI) round with a single-action point detonating fuze was identified as the most probable munition (MPM) that might be encountered in IR Site 2. As shown in Table 3-1, [taken from Table 13-2, Naval Sea Systems Command (NAVSEA), 2001] the Maximum Fragment Throw Range for the 20mm projectile is 320 feet. Based on this range, a 320-foot EZ was established around IR Site 2 and the Possible OEW Burial Site for surface characterization and during the TCRA, respectively. The EZ is shown in Figure 3-2.

All upland areas of IR Site 2, except for the ponds in the wetland areas, were investigated. Until IR Site 2 was cleared of surface OEW, access into the worksite was strictly controlled and limited to UXO-qualified, (or UXO supervised/escorted) authorized, and essential personnel only. The minimum EZ for the OEW characterization was 320 feet. If OEW had been encountered, the EZ would have been expanded to protect other personnel from the blast and fragmentation hazards of accidental detonation of the ordnance type.

The EZ was maintained during the OEW surface sweep operations and TCRA. Access gates were secured, roads were barricaded and posted, and a red "Bravo" flag was flown near the access gates to provide a visual indication of potentially hazardous operations in progress [Foster Wheeler Environmental Corporation (FWENC), 1998]. Procedures were in place for the SUXOS to expand the EZ if OEW was discovered that was unsafe to transport and required blown-in-place procedures.

3.3 EXPLOSIVE SAFETY AND QUANTITY DISTANCE

The MPM identified for the site was the 20mm high-explosive projectile with a net explosive weight (NEW) of 165 grams. Magazine M353 was designated as the explosives storage magazine and had a construction-rated explosive storage limit of 15,000 pounds NEW. For the purposes of the project, the explosives-storage limit was reduced to 500 pounds. The resulting Quantity Distance (Q/D), Inhabited Building Distances (IBD) and Public Transportation Route Distances (PTRD) were within parameters promulgated in Department of Defense (DoD) 6055.9-STD (DoD, 1999). Specifically, the required Q/D and IBD for the NEW limits were at 1,250 feet, the required PTRD was 750 feet, and the potential explosion site (PES) distances

TABLE 3-1

MAXIMUM CASE FRAGMENT RANGES FOR SELECTED SINGLE ITEM DETONATIONS

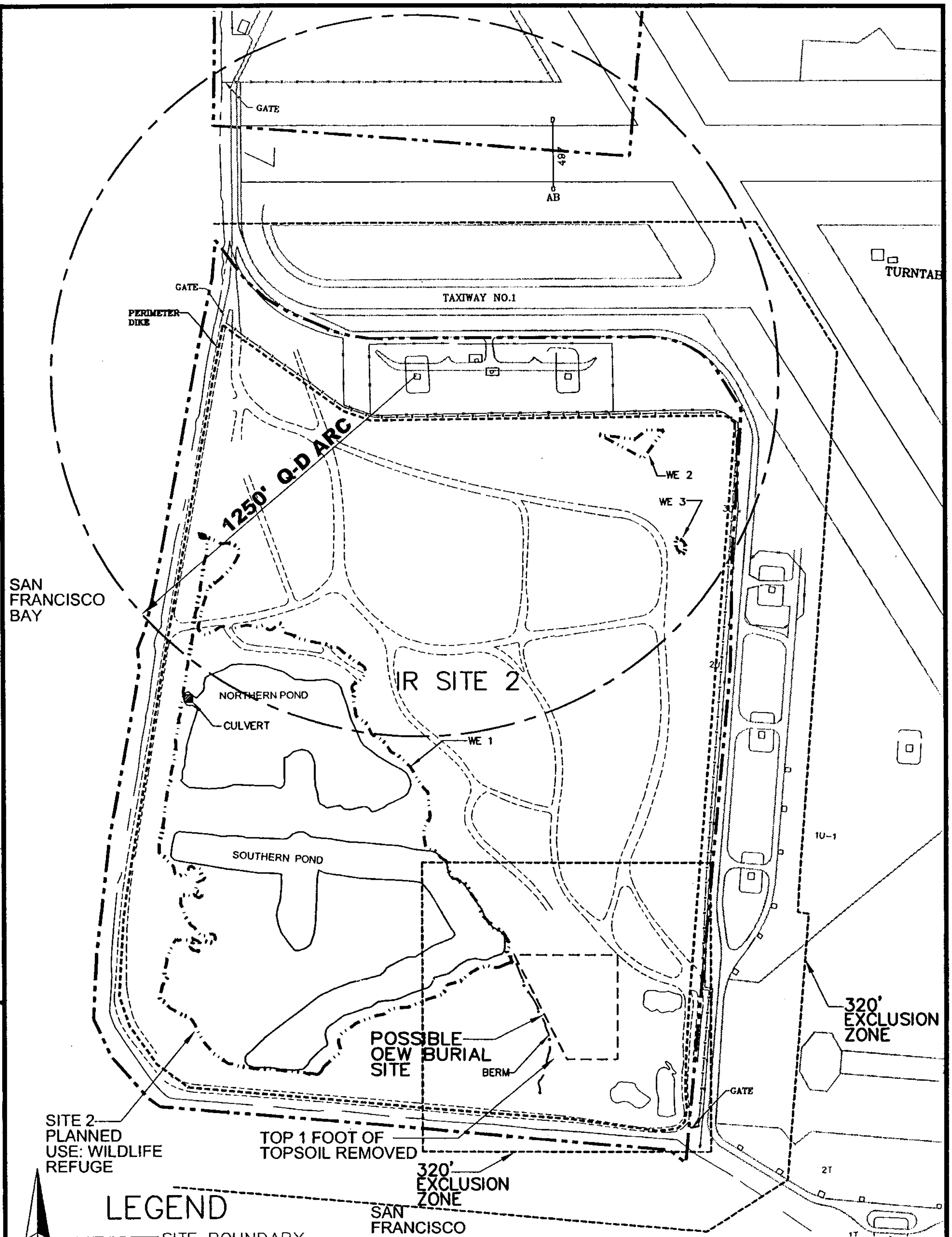
Munition	Maximum Fragment Throw Range (Case Fragments)[1] (feet)
20mm projectile	320
25mm projectile	760
37mm projectile	1,180
40mm projectile	1,100
40mm grenade	345
M 229, 2.75-inch rocket	1,375
M 48, 75mm projectile	1,700
M1, 105mm projectile	1,940
Mk 35, 5-inch/38 projectile	2,205
Mk 64, 5-inch/54 projectile	1,800
M107, 155mm projectile	2,580
M437, 175mm projectile	2,705
M106, 8-inch projectile	3,290
Mk 13 & 14, 16-inch/50 projectile	5,640
M49A3, 60mm mortar	1,080
M374, 81mm mortar	1,235
M3A1, 4.2-inch mortar	1,620
M64A1 500-pound bomb	2,500
Mk 81, 250-pound bomb	2,855
Mk 82, 500-pound bomb	3,180
Mk 83, 1,000-pound bomb	3,290
Mk 84, 2,000-pound bomb	3,880
BLU-109 bomb	4,890

Note:

[1] These calculated fragment throw ranges are for individual items and do not apply to detonations involving multiple rounds.

mm – millimeter
Mk – mark

Source: Naval Sea Systems Command, 2001



LEGEND

SITE BOUNDARY
FENCELINE
EXCLUSION ZONE (320 FT.)
Q-D QUANTITY-DISTANCE ARC (1250 FT.)
PERIMETER DIKE
BERM

N

500 250 0 500

SCALE IN FEET

Figure 3-2
IR SITE 2 EXCLUSION ZONE AND Q-D ARC

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from inhabited buildings and public transportation routes were 1,250 and 750 feet, respectively. Table 3-2 (taken from Table C9.T.1; DoD, 1999) was used to compute the required distances. Figures 3-2 and 3-3 show the various distances and arcs. As shown in Figure 3-3, the nearest inhabited building was well outside the IBD arc, and no other activities were conducted within the Q/D arc. The magazine was secured by a Sargeant-Greenleaf, Model 833 high-security lock, and the gate to the compound was locked as well. The FWENC SUXOS maintained control of the keys to both the magazine and the magazine compound.

A *Final Explosives Safety Remediation Plan* (ESRP) (FWENC, 2002c) was prepared, which provided guidelines for the TCRA activities conducted in the Possible OEW Burial Site. These activities are addressed in the *Final Time-Critical Removal Action Closeout Report* (FWENC, 2002a).

3.4 OEW AVOIDANCE PROCEDURES

OEW avoidance procedures were used for all intrusive exploration including test pits, sample borings, and cone penetration tests (CPTs). Schonstedt downhole and Schonstedt GA-52 CX magnetometers were used to locate and avoid UXO during the intrusive activities.

3.4.1 Test Pits

UXO technicians cleared each test pit location of metal debris by scanning the area with the Schonstedt magnetometer. After finding a location the magnetometer indicated was free of detectable metal, the soil was mechnically removed in 1-foot lifts. UXO technicians checked the test pit with the magnetometer after each lift. Metal detected within 1 foot of the surface was hand-excavated to determine if it was OEW. This process was repeated until the required test pit depth was reached. Section 4.2.2 discusses test pit findings and includes a summary table of test pit exploration findings.

3.4.2 Boreholes

UXO technicians cleared each borehole location of metal debris. After finding a location a magnetometer indicated was free of detectable metal, the borehole was started with a hand-held auger. At a depth of 6 inches, the magnetometer probe was inserted into the borehole and checked for metal. This procedure was repeated every 6 inches until the maximum depth of the hand-held auger was reached at approximately 4 feet. If the borehole was clear of metal debris, the SUXOS would approve mobilization of drilling equipment and supplies to the borehole location. The drill rig was then positioned over the borehole and augered down to a maximum depth of 8 feet. The drilling string was pulled, the drill rig was relocated to a position at least 20 feet away from the borehole, and the magnetometer probe was lowered into the borehole to check for metal. This procedure was repeated every 4 feet until a depth of 20 feet was reached, or until the first sampling depth (less than 20 feet) was reached. After reaching 20 feet, OEW

TABLE 3-2

INHABITED BUILDINGS AND PUBLIC TRAFFIC ROUTE DISTANCES

Net Explosive Weight lbs	Distance in Feet to Inhabited Building From:				Distance in Feet to Public Traffic Route From:			
	Earth-covered Magazine			Other PES	Earth-covered Magazine			Other PES
	Front	Side	Rear		Front	Side	Rear	
1	500	250	250	1,250	300	150	150	750
2	500	250	250	1,250	300	150	150	750
5	500	250	250	1,250	300	150	150	750
10	500	250	250	1,250	300	150	150	750
20	500	250	250	1,250	300	150	150	750
30	500	250	250	1,250	300	150	150	750
40	500	250	250	1,250	300	150	150	750
50	500	250	250	1,250	300	150	150	750
100	500	250	250	1,250	300	150	150	750
150	500	250	250	1,250	300	150	150	750
200	700	250	250	1,250	420	150	150	750
250	700	250	250	1,250	420	150	150	750
300	700	250	250	1,250	420	150	150	750
350	700	250	250	1,250	420	150	150	750
400	700	250	250	1,250	420	150	150	750
450	700	250	250	1,250	420	150	150	750
500	1,250	1,250	1,250	1,250	750	750	750	750
600	1,250	1,250	1,250	1,250	750	750	750	750
700	1,250	1,250	1,250	1,250	750	750	750	750
800	1,250	1,250	1,250	1,250	750	750	750	750
900	1,250	1,250	1,250	1,250	750	750	750	750
1,000	1,250	1,250	1,250	1,250	750	750	750	750
1,500	1,250	1,250	1,250	1,250	750	750	750	750
2,000	1,250	1,250	1,250	1,250	750	750	750	750
3,000	1,250	1,250	1,250	1,250	750	750	750	750
4,000	1,250	1,250	1,250	1,250	750	750	750	750
5,000	1,250	1,250	1,250	1,250	750	750	750	750
6,000	1,250	1,250	1,250	1,250	750	750	750	750
7,000	1,250	1,250	1,250	1,250	750	750	750	750
8,000	1,250	1,250	1,250	1,250	750	750	750	750
9,000	1,250	1,250	1,250	1,250	750	750	750	750
10,000	1,250	1,250	1,250	1,250	750	750	750	750
15,000	1,250	1,250	1,250	1,250	750	750	750	750

Notes:

PES – potential explosion site
lbs – pounds

Source: Naval Sea Systems Command, 2001

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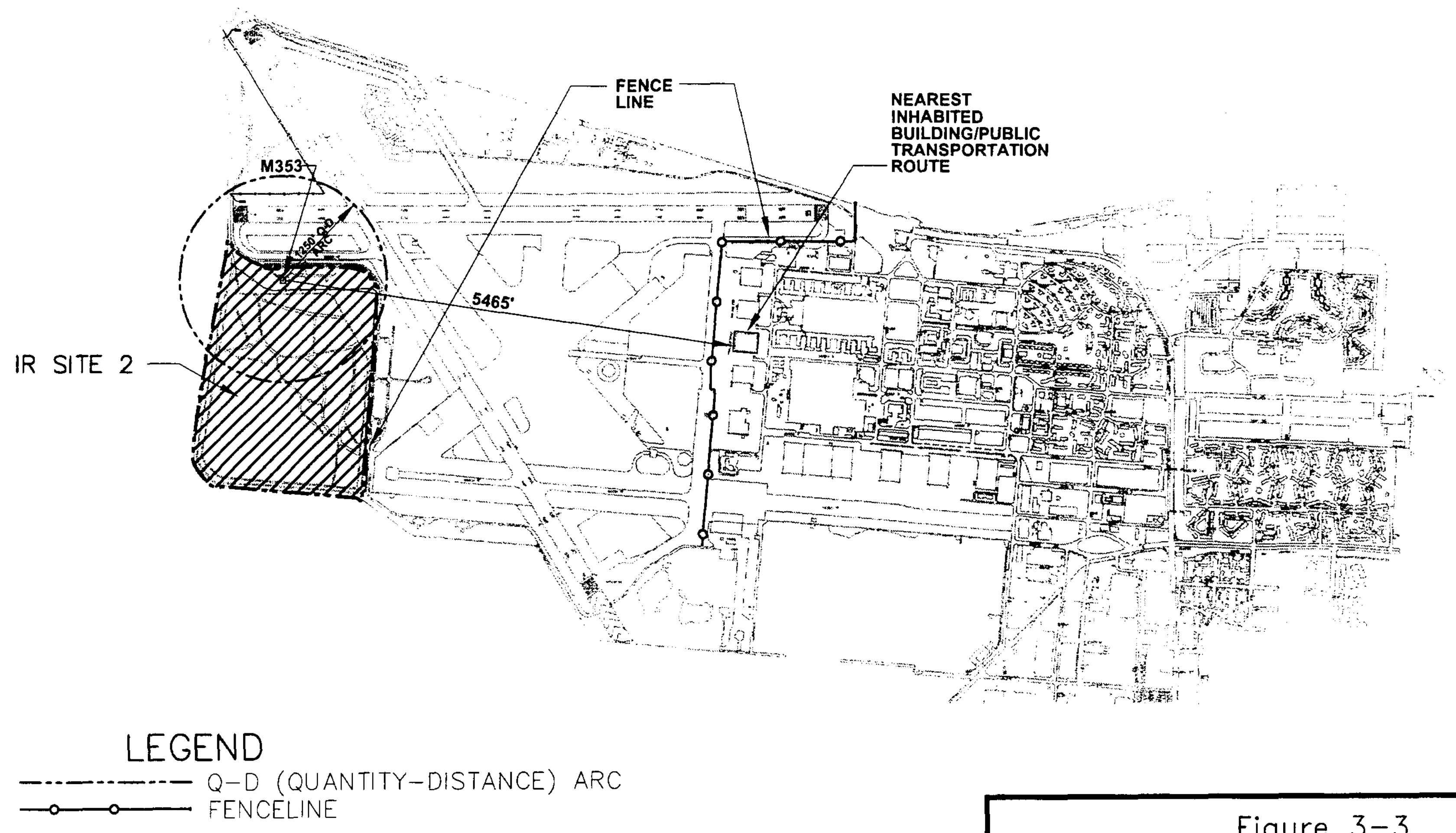

Figure 3-3
QUANTITY-DISTANCE FOR IR SITE 2

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Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

avoidance procedures were suspended and drilling proceeded to sampling depth. If metal was detected early in the boring, the drill rig was relocated to an alternate site and the process was repeated. If metal contamination was found before reaching a depth of 20 feet, drilling ceased, and the decision to continue or relocate the boring location was evaluated by the SUXOS and field engineer/geologist.

3.5 QC PROCEDURES

The project QC team was comprised of the UXO QC Representative (USACE quality assurance (QA)/QC-certified), the SUXOS, the Project Quality Control Manager (PQCM) and the Project Manager. All were responsible for implementing QC procedures contained in the Contractor Quality Control (CQC) Plan, which was an appendix of the *Final Focused Remedial Work Plan* (Work Plan) (FWENC, 2002b). All distinguishable aspects of the project that required measures to verify the quality of work performed and compliance with specified requirements were identified as definable features of work (DFWs), and controls for each DFW were assigned. The CQC Plan (FWENC, 2002b) implemented preparatory, initial and follow-up control phases for all aspects of every DFW.

The Southwest Division Naval Facilities Engineering Command (SWDIV) QA Officer reviewed the CQC Plan (FWENC, 2002b) to ensure that it was in compliance with the requirements of Naval Facilities Engineering Command (NAVFAC) P-445 [Construction Quality Management (CQM) Program] (NAVFAC, 2000), Unified Facilities Guide Specification (UFGS)-D 01450H (NAVFAC, 2003). Changes were made to the latest revision of NAVFAC P-445 to bring it and UFGS-D 01450H into agreement. The QA Officer was required to approve the CQC Plan (FWENC, 2002b) prior to its implementation. SWDIV recommendations for improvements to the CQC Plan (FWENC, 2002b) were incorporated into the draft version of the plan, and it was further refined during the review process.

Additional Navy oversight of the QC process was provided by the Naval Ordnance Safety and Security Activity (NOSSA) who reviewed the CQC Plan (FWENC, 2002b), the Work Plan (FWENC, 2002b), and the Action Memorandum. Their comments and recommendations were incorporated into the documents.

As a part of SWDIV QA oversight, the Resident Officer in Charge of Construction (ROICC) was notified prior to the administration of every Search and Effectiveness Probability (SEP) test so that ROICC or a staff member could observe the test-grid preparation and conduct of the evaluation. Additionally, the SEP tests and other portions of the CQC Plan (FWENC, 2002b) that affected other aspects of ongoing site activities were discussed during weekly CQC meetings between the ROICC, Remedial Project Manager (RPM), Environmental Compliance Manager (ECM), and the Contractor. These meetings were held to further ameliorate the QA/QC process by identifying elements of the plan that could be modified to optimize the realized results.

Each UXO team conducting surface clearance operations was certified in the surface QC test grid using the SEP test. To gain certification in surface clearance operations, each surface clearance team was required to demonstrate the ability to achieve an 85 percent probability of detection (PD) with a 90 percent confidence level of removal of target items. The cumulative binomial probability was applied in determining 85 percent PD at a 90 percent confidence level.

A surface QC test grid was established and seeded with 34 target items that were representative of the target items being searched for (20mm projectiles). A mixture of inert UXO items and fragments were used to seed the surface QC test grid. To achieve 85 percent PD at a 90 percent confidence level, 32 of the 34 target items were required to be located by the each team in the surface QC test grid. If less than 32 items were located, the UXO team was required to continue training until they achieved the 85 percent PD at a 90 percent confidence level.

When new team members who had not successfully completed the SEP certification were added to the UXO team, the entire team was required to reprocess through the surface QC test grid and demonstrate the ability to achieve an 85 percent PD at a 90 percent confidence level before continuing field operations.

Establishing the surface QC test grid and processing teams through the surface QC test grid were functions of QC and remained separate and independent from other operations.

After certifying and documenting the successful certification of each UXO team to conduct surface clearance operations, SEP tests were conducted periodically for each team to monitor the continued effectiveness of surface clearance operations. Initially, SEP tests were performed twice a month for each UXO team. The frequency of these tests was based upon the performance of the individual teams. This determination was made by the Project Manager, or SUXOS with concurrence of the Site UXO QC Representative. The objective for the surface clearance remained at 85 percent PD with 90 percent confidence level of removal.

Periodic SEP tests were conducted which involved QC personnel selecting a SEP test grid from the daily scheduled grids. The selected grid was seeded with a predetermined number of target items, which were marked as QC SEP test items. After the UXO team completed surface clearance operations in the selected grid, all QC test items are separated from other items recovered. The QC personnel then determined if the number of SEP test items recovered was sufficient to achieve the 85 percent PD with 90 percent confidence level criteria. Failing to achieve this, the team was decertified from conducting surface clearance operations. The team's search techniques were then examined to identify the cause for failure, and corrective action was initiated. After corrective action was applied, the decertified team was tested again prior to resuming surface clearance operations.

3.6 LINER INSTALLATION

A protected nesting site for the endangered California least tern (*Sterna antillarum*) is located on the runway tarmac approximately 1 mile east (inland) from IR Site 2. The excavation of the 2.5-acre Possible OEW Burial Site and the associated grading activities that were conducted on the 4.5-acre (approximate) area resulted in a complete removal of vegetation. FWENC and Navy biologists observed the area and determined that it created a potential nesting habitat similar to that preferred by the California least terns. This condition was found untenable because of the feral cat and raptor (American peregrine falcons, red-tailed hawks) populations that are established at IR Site 2.

As a remedy, the area was covered with a dark-colored liner measuring approximately 35,000 square yards. The liner used was a 12-mil-thick, high-strength polyethylene reinforced with Skrim. Used tires, modified specifically for use as silage ballast, were selected for anchoring the liner.

A liner anchor trench, 1 to 1½ feet in depth, was excavated. Because a portion of the liner would be placed in the Possible OEW Burial Site excavation area, UXO avoidance procedures were followed when excavating the anchor trench. UXO technicians checked the marked excavation lines with a Schonstedt ordnance locator prior to beginning excavation activities. The soil was removed in 6-inch lifts. UXO technicians continuously checked the trench before each cut to ensure that OEW was not encountered.

After completing the liner anchor trench, the liner was delivered to the project site in four sheets. A crew of eight deployed the liner sheets. The workers spaced themselves along the length of the leading side of the liner sheet being installed and traversed the site, unfolding the liner behind them. When they reached the anchor trench on the opposite side of the excavation, the liner edge was placed in the trench and ballast was applied. The modified tires were then spread in rows across the liner surface, approximately 5 feet apart, north-to-south, and 10 feet apart, east-to-west. These procedures were repeated until all four of the sheets had been anchored in the perimeter anchor trenches and the modified tire ballast had been distributed in the pattern recommended by the manufacturer. A liner seam-stitching machine was used to attach abutting liner seams together. Then, the anchoring trenches were completely backfilled and the equipment demobilized from the site. A final acceptance inspection of the site was conducted by the Navy ROICC, and the installation was considered complete.

Removal of the liner was conducted at the conclusion of the nesting season, and the area was hydroseeded.

3.7 OEW CHARACTERIZATION RESULTS

A 20-foot by 20-foot grid network was installed on the Possible OEW Burial Site (Figure 3-4) to document the location of subsurface OEW. One M56 anti-tank/anti-personnel (AT/AP) inert land mine and one 20mm target practice projectile were found during the surface characterization of IR Site 2. An additional 8,675 target practice projectiles were uncovered during the TCRA. None of the OEW encountered contained any explosives or energetics. Figure 3-5 shows the location of recovered OEW. The OEW was documented on the UXO Acquisition and Accountability Log forms provided as an attachment to the Standard Operating Procedure (SOP)-1 (FWENC, 2002b) for OEW disposal. Copies of these forms with the appropriate information are located in Appendix F of this document. Recovered OEW was accumulated in Magazine M353 as the surface characterization and TCRA were conducted. After characterization and removal activities were completed, all 20mm rounds were demilitarized in accordance with the DoD Defense Material Disposition Manual 4160.21-M-1, which called for cutting each projectile in half. This was accomplished by using an electric reinforcing-bar cutter. The demilitarized rounds were shipped to a Class III landfill facility for disposal as non-hazardous scrap steel. Photograph 1 in Appendix G shows 20mm rounds being demilitarized.

The M56 AT/AP inert land mine was transferred to the Navy EOD Detachment Southwest Unit at Building 41, Naval Air Station North Island. A copy of the e-mail confirming transferal has been included in Appendix F.

The OEW characterization and subsequent TCRA conducted at IR Site 2 verified the presence of OEW on and below the ground surface within the Possible OEW Burial Site. Uncertainties exist as to the types of OEW material buried in the landfill. When the Final Feasibility Study is promulgated, information concerning appropriate land use controls for the site will be provided as part of the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) process, that is, development of the Proposed Plan and Record of Decision. Engineering and institutional controls will be established to address the landfill cap placement and construction, and any excavation below the current land surface to mitigate potential risks associated with intrusive activities.

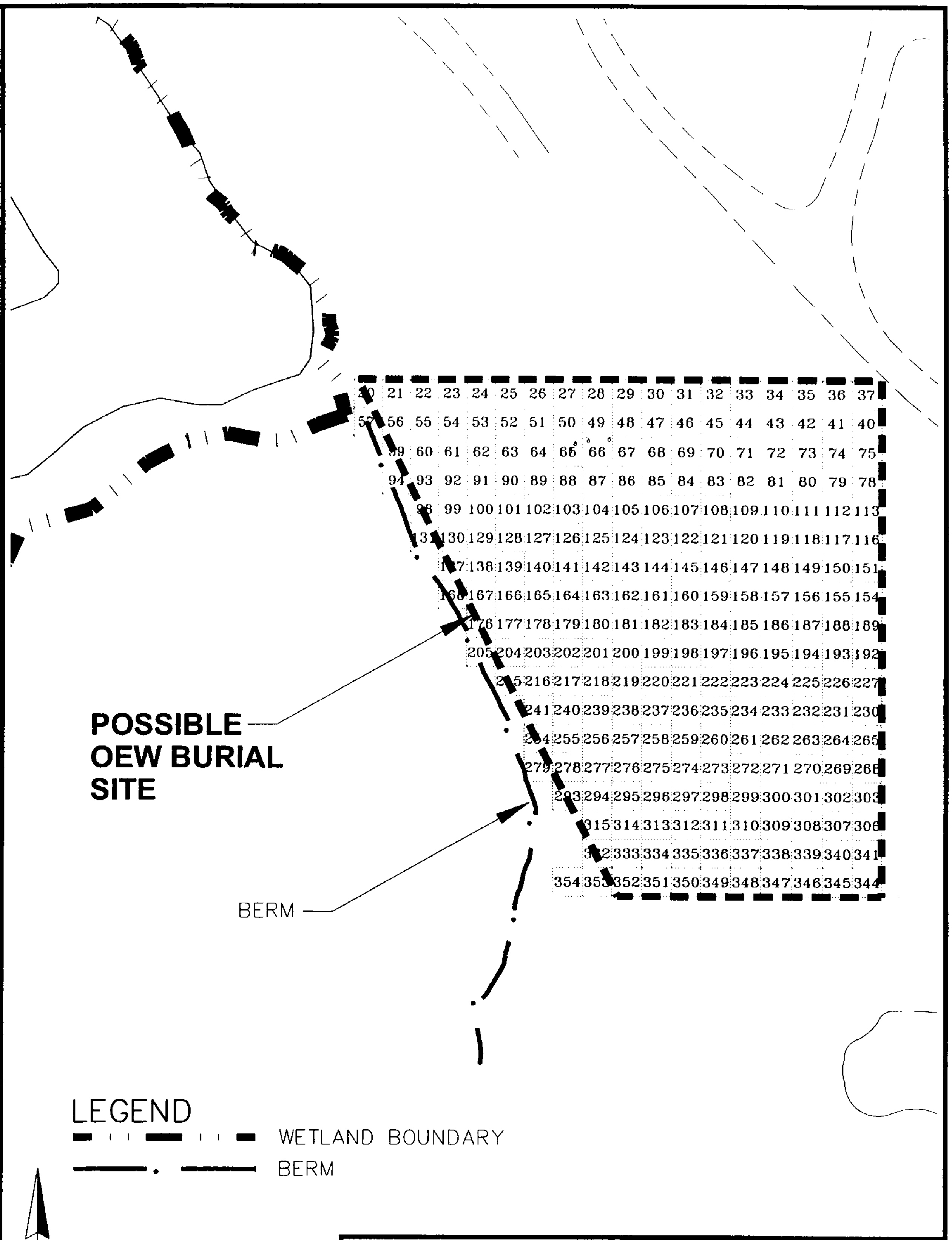


Figure 3-4
POSSIBLE OEW BURIAL SITE GRID MAP
ALAMEDA, CA

Southwest Division
Naval Facilities Engineering Command

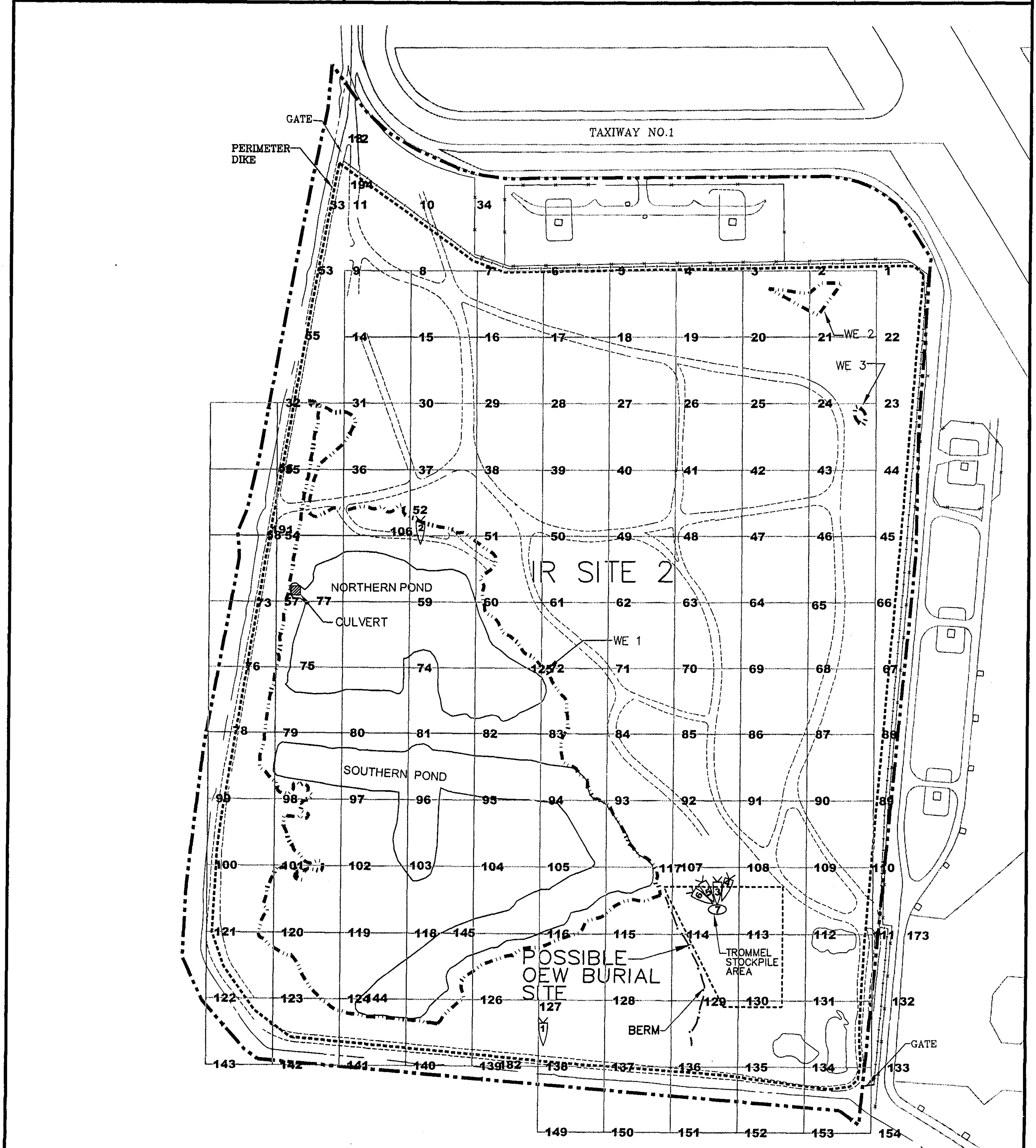
FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWN BY: MD | DATE: 10/29/03
CHECKED BY: TL | REV: REVISION 1
APPROVED BY: AL
DCN: FWSD-RAC-03-2899 | CTO: #0054
DRAWING NO: 03289934.DWG

I:\1990-RAC\CTO-0054\DWG\032899\03289934.DWG
PLOT/UPDATE: SEP 16 2003 08:12:24

I:\1990-RAC\CTO-0054\DWG\033604\03360435.DWG
PLOT/UPDATE: DEC 10 2003 13:28:37

DRAWN BY: MD	CHECKED BY: TL	APPROVED BY: AL	DCN: FWSD-RAC-03-3604	DRAWING NO: 03360435.DWG
DATE: 12/24/03	REV: REVISION 0		CTO: #0054	



LOCATION #	ITEMS FOUND
1	1–20 MM TP ROUND
2	1–AT/AP LANDMINE
3	842–20 MM TP ROUNDS
4	5021–20 MM TP ROUNDS
5	96–20 MM TP ROUNDS
6	1657–20 MM TP ROUNDS
7	725–20 MM TP ROUNDS

NOTE: THE TIP OF THE SYMBOL SHOWS THE SURVEYED LOCATION.

LEGEND
- SITE BOUNDARY
- FENCELINE
- WETLAND BOUNDARY
- BERM
- PERIMETER DIKE
- WE WETLAND

300 150 0 300
SCALE IN FEET

Figure 3–5
OEW LOCATIONS ON IR SITE 2

Southwest Division
Naval Facilities Engineering Command

TETRA TECH FW, INC.

4.0 GEOTECHNICAL AND SEISMIC EVALUATIONS

Summaries of the geotechnical and seismic evaluations performed are presented in this section. This section provides details pertaining to background geologic features, summary of field exploration and testing activities and results, data interpretation, geotechnical engineering analyses, and seismic hazards evaluation. Issues that are addressed include geotechnical characteristics of the existing soil cover, subsurface strata features, liquefaction potential, expected earthquake-induced settlements and lateral deformations, immediate and long-term settlements from a proposed landfill cap, and stability of slopes. Hushmand Associates, Inc. (HAI) assisted Foster Wheeler Environmental Corporation (FWENC) in performing the geotechnical and seismic evaluations (Attachment 1).

4.1 FIELD EXPLORATION AND TESTING

Field explorations were conducted in accordance with Section 4.6 of the *Final Focused Remedial Investigation Work Plan* (Work Plan) (FWENC, 2002b). All field activities were conducted in accordance with established data quality objectives (DQOs) (see Tables 1-2 and 1-3). The activities included optimizing the locations of the sampling points, refining the field investigations as they were performed, and developing the laboratory testing program. The purpose of these explorations was to collect soil samples and data in order to perform geotechnical and seismic hazard evaluations at Installation Restoration (IR) Site 2 and in the Additional Investigation Area between IR Sites 1 and 2. Fieldwork at IR Site 2 started on February 8, 2002, and was completed on March 15, 2002. Fieldwork in the area between IR Sites 1 and 2 was performed from June 19, 2002, through June 29, 2002. Photographs 6 through 16 in Appendix G show field exploration activities and equipment used.

The field investigations included the following tasks:

- Cone penetration test (CPT) soundings
- Test pit explorations
- Borehole sampling

A total of 21 CPTs, 12 test pit explorations, and 15 soil borings were conducted in the study area. The locations of the sampling were selected to coincide with the proposed transects, which were used later to develop cross sections for stability analyses. Upland sampling locations were marked and surveyed by Kister, Savio, and Rei, Inc. (KSR), while the offshore soil boring locations were surveyed by FWENC. All sampling locations are shown in Figure 2-1. Samples obtained from field explorations were forwarded to Teratest Labs, Inc. (Teratest) (on March 18, 2002, for IR Site 2 and then on July 9, 2002, for the Additional Investigation Area between IR Sites 1 and 2) for geotechnical testing.

4.1.1 Cone Penetration Testing

Fifteen exploratory electronic CPT soundings were performed by Holguin, Fahan, & Associates, Inc. (HFA) within the limits of IR Site 2 and were supervised by HAI. An additional six CPT soundings were performed by Gregg Drilling and Testing, Inc., in the Additional Investigation Area between IR Sites 1 and 2. These tests were conducted using a 20-ton CPT rig in accordance with American Society for Testing and Materials (ASTM) Test Method D 3441 and D 5778. The CPTs were located along the shoreline on the southern and western perimeter of the site at an approximate 150- to 250-foot spacing. The tests yielded an approximately continuous representation of the soil conditions and in situ strength parameters. No soil samples were retrieved during CPT testing.

CPT data was collected by pushing an instrumented cone-tipped probe into the soil while simultaneously recording the tip resistance and side (sleeve) friction resistance of the soil during penetration. The 21 CPT soundings included pore water pressure measurements to more clearly define stratigraphic conditions in terms of thickness and penetration resistance of subsurface soils. The CPT data processing was performed using a computer-based data acquisition and presentation system.

The planned depth of the CPT varied for different locations at the site. In accordance with the requirement of the Work Plan (FWENC, 2002b), the planned depths of CPTs were anticipated to be approximately 60 feet below ground surface (bgs). This depth was determined to be sufficient to investigate the predominant soil layers that could influence liquefaction potential evaluation and slope stability analyses. However, due to the presence of the wetland area and narrow coastal margin, most of the CPTs were performed on top of a 10-foot-high (approximate) berm. To account for the height of the berm, the planned depth of the CPTs were increased to 75 feet below the top of the berm. In addition, preliminary CPT results at Location C-2-15A confirmed the presence of a deep Young Bay Mud layer at the northern boundary of IR Site 2. Because of the potential effects on site stability, additional CPTs were performed in the area between IR Sites 1 and 2. The target depths of these additional CPTs were up to 200 feet. Table 4-1 provides identification of CPTs, corresponding depths, and surface elevation of the berm (when applicable).

The sampling location designations used in the report may not coincide with the designation used by the surveyors or those reported in the logs. Therefore, cross-references between the "Sample Location ID #" (designation used in the text and figures) and its corresponding "Survey Point Number" (used by the surveyors) and "CPT location ID #" (as recorded in the CPT logs) are included in Table 4-1.

In addition to collecting standard CPT data, seismic wave velocities were directly measured in accordance with ASTM Test Method D 4428/D 4428M at several locations. The seismic wave velocities were used to determine dynamic soil parameters used in the seismic phase of the

TABLE 4-1

CPT SURFACE ELELVATION AND TOTAL DEPTHS

CPT Number	Survey Point Number	CPT Location ID#	Total Depth Drilled (feet)	Elevation[1]
C-2-1	1002	CPT-01	67.75	16.55
C-2-2	1003	CPT-02	75.46	16.98
C-2-3	1004	CPT-03	75.62	16.12
C-2-4	1007	CPT-04	75.79	15.70
C-2-5	1008	CPT-05	71.69	15.84
C-2-6	1010	CPT-06Seis	85.14	16.12
C-2-7	685	CPT-07	73.98	16.50
C-2-8	1012	CPT-08	75.62	16.14
C-2-9	1014	CPT-09	76.28	15.88
C-2-10	1015	CPT-10	76.28	13.06
C-2-11E	1016	CPT-11	60.53	5.08
C-2-12A	1017	CPT-12A	76.28	6.34
C-2-13	1018	CPT-13Seis	84.15	6.41
C-2-14	1019	CPT-14	63.48	4.45
C-2-15A	1021	CPT-15A	141.08	6.84
C-2-16	757	CPT-757	194.38	4.92
C-2-17	758	CPT-758	183.72	4.36
C-2-18	752	CPT-752	150.10	6.44
C-2-19	753	CPT-753	189.13	7.93
C-2-20	750	CPT-750	150.10	7.52
C-2-21	751	CPT-751	190.12	9.18

Notes:

[1] Elevation measured in feet above mean sea level

CPT - cone penetration test

evaluation. The planned depth of these measurements at IR Site 2 was around 100 feet bgs. Seismic wave velocities were measured at CPT Locations C-2-6 and C-2-13 to depths of 85 (from top of berm approximately 10 feet above adjacent site surface) and 84 feet below the adjacent site surface, respectively. Due to high-tip resistance encountered at these depths, the seismic wave velocity measurements from 84 to 100 feet bgs were not conducted. Instead of performing additional CPTs, the data gaps were filled by using data from two deep upland soil borings. In the area between IR Sites 1 and 2, the seismic test was conducted at CPT Location C-2-19 and had no difficulty reaching the target depth of approximately 100 feet.

Results of the CPTs were used to select the types of test [standard penetration test (SPT), drive sampling, or push sampling] and depths for each soil boring.

4.1.2 Test Pit Exploration

A total of 12 test pits were excavated as indicated in the Work Plan (FWENC, 2002b). The results of the test pit explorations were used to characterize the thickness and composition of the current cover over the landfill. Since the test pits were located within landfill areas, metal avoidance procedures, similar to OEW avoidance procedures, as discussed in Section 3.4, were followed. A backhoe with a 24-inch-wide bucket was used for excavation. All test pits were excavated to a maximum of 4 feet bgs or until waste material was encountered. Bulk soil samples, weighing approximately 20 pounds, were collected for geotechnical soil testing.

Materials encountered during test pit excavations were logged by a FWENC field engineer/geologist. Data collected and provided in the logs include: test pit number, location, ground surface elevation, excavation area and depth, description and notes, and profile of pit wall. Field visual soil classification of soil samples was conducted according to ASTM Test Method D 2488 procedures (Standard Practice for Description and Identification of Soils, Visual-Manual Procedure). Munsell soil color charts were used to define the soil colors. Soil moisture content and any significant water flow was recorded. In addition, any waste or odors encountered during sampling was noted.

4.1.3 Soil Borings

Upland and offshore borehole sampling was performed to obtain soil penetration resistance data and soil stratification information. Samples and data collected included disturbed and relatively undisturbed samples from SPT and Modified California (MC) drive samplers, relatively undisturbed direct-push samples using thin-walled Shelby tube sampler, and blow counts from SPT and MC samplers. Sampling was performed in general accordance with the following ASTM standard test methods:

- ASTM Test Method D 1586-99, Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils

- ASTM Test Method D 3550-01, Standard Practice for Thick Wall, Ring-Lined, Split Barrel, Drive Sampling of Soils
- ASTM Test Method D 1587-00, Standard Practice of Thin-Walled Tube Sampling of Soils for Geotechnical Purposes

The sampling method, interval, and depths of sampling were determined using results from CPTs. In general, sampling was performed at 5-foot intervals, which is sufficient to yield a continuous characterization of the soil. Drive samples (from SPT and MC samplers) were generally taken at depth intervals with relatively high cone-tip resistance and low friction ratio as measured in CPTs. These intervals normally contained coarse-grained soils. In addition, direct-push samples (Shelby tubes) were obtained at depth intervals with relatively low tip resistance (silty and clayey soils) and high friction ratio. SPT and sampling were conducted using an unlined split-spoon sampler (1.375-inch inside diameter and a 2.0-inch outside diameter) in accordance with ASTM Test Method D 1586 and driven by a 140-pound hammer falling 30 inches. High quality soil samples of fine-grained soils were obtained using hydraulically pushed thin-walled Shelby tubes (3-inch inside diameter) in accordance with ASTM Test Method D 1587, at locations selected by the geotechnical engineer in the field.

Soil samples were also collected using a MC drive sampler (2 or 2.5-inch inside diameter and a 2.75-inch outside diameter) lined with brass rings or tubes in accordance with ASTM Test Method D 3550, and driven by a 140-pound hammer falling 30 inches. Exploratory drilling activities were conducted under the direction of a FWENC field engineer, who collected the soils samples and logged the borings in accordance with ASTM Test Method D 2488, Standard Practice for Description and Identification of Soils, Visual-Manual Procedure.

A total of nine upland borings at Locations B-2-7A through B-2-15 and six offshore borings at Locations B-2-1 through B-2-6 (see Figure 2-1) were performed. Upland borings at IR Site 2 and the Additional Investigation Area between IR Sites 1 and 2 were conducted by Gregg Drilling and Testing, Inc. and Western Strata Exploration, respectively. A FWENC field engineer/geologist was present to coordinate the borings and to collect/record samples. Gregg Drilling and Testing, Inc., utilized a Mobile B-80 mud rotary drill rig while Western Strata Exploration used a Mobile B-61 mud rotary drill rig for the upland borings. Sampling locations were surveyed by KSR. The offshore borings were conducted by Gregg Drilling and Testing, Inc., using a Mobile B-80/22 mud rotary drill rig set up on the boat, *Quin Delta*. FWENC personnel were on board to survey the sampling locations, monitor the borings, record logs, and collect soil samples. A Trimble Ag132 (accurate to +/- 3 feet) digital global positioning system (DGPS) was used for navigation and to record the actual horizontal coordinates at each boring location. The mud line elevation at each boring location was determined by measuring water depth with a lead line and correcting the depth to a corresponding elevation. A tide gauge (Herrin Design 3011), which transmitted water surface elevation data to the *Quin Delta,* was installed on the "T" pier in the Oakland Inner

Harbor entrance channel. Vertical control was based on Site Control No. 500 (see Figure 2-2). Table 4-2 lists the actual coordinates for each boring location.

Materials encountered during the drilling activities for both upland and offshore soil borings were logged by a FWENC field engineer/geologist. Data collected in the logs included: boring number, location, ground surface elevation, excavation depth, date drilled, sampling method, individual who logged data, sample interval, sample number, blow counts, sample recovery percent, soil classification, and description and remarks. Field classification of soil samples was made according to ASTM Test Method D 2488 procedures (Standard Practice for Description and Identification of Soils, Visual-Manual Procedure). Munsell soil color charts were used to identify the soil colors. In addition, any waste or odors encountered while sampling were noted.

4.1.4 Laboratory Testing

Samples obtained from field explorations were forwarded to Teratest for testing. A chain-of-custody (COC) record was filled out for all samples to facilitate handling and transportation. COC procedures detailed in Section 4.6.6.1 of the Work Plan (FWENC, 2002b) were followed. All COC records are included in Appendix H.

An initial laboratory testing program, was conducted according to the Navy-approved Work Plan (FWENC, 2002b), which details the tests, testing method, sample type, and approximate number of tests to be performed. This laboratory test program was managed by HAI with oversight from FWENC. HAI optimized the laboratory testing program by evaluating field results and initial laboratory test results and analyses. The majority of samples selected for testing and types of tests chosen were based on evaluating CPT data, soil boring, and trench logs. Further testing was requested while developing cross sections for analyses and performing preliminary calculations. Field Change Requests (FCRs) describing modifications to the test program described above are included in Appendix I.

Two tests listed on the original testing program were not performed. These included the Modified Proctor test to determine compaction characteristics (ASTM Test Method D 1557) and saturated hydraulic conductivity tests (ASTM Test Method D 5084). Both tests were to be performed on bulk samples from the existing cover soil. The tests were designated "if needed" in the original Work Plan (FWENC, 2002b) and were determined not be necessary. Field explorations were conducted to investigate the feasibility of using the existing cover soil as part of a final cover design. Results from test pit explorations showed that the material was not suitable. Therefore, both tests involving the existing cover soils were not needed. Additional discussion is presented in Section 4.5.2, Hydraulic Performance of Existing Soil Cover.

The soil samples sent to Teratest included 20-pound bulk samples obtained from test pits, 3-inch by 30-inch Shelby tubes, and 2-inch by 6-inch sleeves from soil borings. Tests performed include Atterberg Limits, particle size analyses (with hydrometer), unconsolidated-undrained (UU)

TABLE 4-2

SURVEY COORDINATES OF SAMPLE LOCATIONS

Sample Location ID#	Survey Point Number	CPT Location ID#	Northing	Easting	Elevation[1]
B-2-1	B-2-1[2]	N/A	471492.67	1472681.90	51.50
B-2-2	B-2-2[2]	N/A	471580.26	1471885.60	61.50
B-2-3	B-2-3[2]	N/A	471781.44	1470839.30	56.50
B-2-4	B-2-4[2]	N/A	472622.24	1470677.30	61.50
B-2-5	B-2-5[2]	N/A	473552.55	1470910.90	81.50
B-2-6	B-2-6[2]	N/A	474566.76	1471014.80	66.50
B-2-7	1000	N/A	471881.36	1472780.20	16.85
B-2-7A	1001	N/A	471897.59	1472721.18	8.53
B-2-8	1006	N/A	471793.07	1471774.97	16.77
B-2-9	1009	N/A	471842.92	1471188.74	16.23
B-2-10	1013	N/A	472570.08	1470959.53	16.78
B-2-11	1022	N/A	474454.47	1471417.73	7.66
B-2-12	1020	N/A	473453.12	1471403.97	4.57
B-2-13	756	N/A	474730.95	1471273.90	4.97
B-2-14	754	N/A	474877.00	1471436.08	7.16
B-2-15	755	N/A	474903.96	1471652.05	7.55
C-2-1	1002	CPT-01	471714.16	1472726.31	16.55
C-2-2	1003	CPT-02	471724.17	1472551.35	16.98
C-2-3	1004	CPT-03	471779.58	1471948.14	16.12
C-2-4	1007	CPT-04	471807.01	1471601.92	15.70
C-2-5	1008	CPT-05	471825.97	1471353.29	15.84
C-2-6	1010	CPT-06Seis	471890.49	1471071.98	16.12
C-2-7	685	CPT-07	472065.99	1470898.20	16.50
C-2-8	1012	CPT-08	472313.73	1470918.73	16.14
C-2-9	1014	CPT-09	472644.14	1470972.15	15.88
C-2-10	1015	CPT-10	473010.44	1471035.29	13.06
C-2-11E[3]	1016	CPT-11	473576.93	1471073.24	5.08
C-2-12A	1017	CPT-12A	473912.53	1471133.24	6.34
C-2-13	1018	CPT-13Seis	474177.97	1471234.19	6.41
C-2-14	1019	CPT-14	473460.93	1471409.91	4.45
C-2-15A	1021	CPT-15A	474512.54	1471266.68	6.84
C-2-16	757	CPT-757	474710.01	1471282.03	4.92
C-2-17	758	CPT-758	474686.08	1471480.03	4.36
C-2-18	752	CPT-752	474913.92	1471259.03	6.44
C-2-19	753	CPT-753	474924.97	1471431.99	7.93
C-2-20	750	CPT-750	475121.98	1471256.92	7.52
C-2-21	751	CPT-751	475110.99	1471446.98	9.18

TABLE 4-2

SURVEY COORDINATES OF SAMPLE LOCATIONS

Sample Location ID#	Survey Point Number	CPT Location ID#	Northing	Easting	Elevation[1]
TP-2-1	666	N/A	471853.35	1472682.76	9.28
TP-2-2	1005	N/A	471851.57	1472332.76	8.39
TP-2-3	663	N/A	472542.10	1472735.10	9.48
TP-2-4	664	N/A	472581.08	1472228.51	13.18
TP-2-5	659	N/A	473332.75	1471767.37	6.57
TP-2-6	658	N/A	473377.52	1472338.54	8.34
TP-2-7	657	N/A	473330.83	1472857.12	11.74
TP-2-8	662	N/A	473601.31	1471252.62	4.15
TP-2-9	654	N/A	473991.83	1471836.37	5.98
TP-2-10	655	N/A	474003.87	1472375.30	5.38
TP-2-11	653	N/A	474315.71	1471436.68	8.76
TP-2-12	656	N/A	474067.01	1472892.43	6.21

Notes:

[1] Elevation measured in feet above mean sea level

[2] Foster Wheeler Environmental Corporation survey ID for offshore borings

[3] Subcontractor survey ID lists as C-2-11

CPT - cone penetration test

N/A - not applicable

triaxial shear test, consolidated-undrained (CU) triaxial test with pore pressure measurements, consolidation tests, water content, percent passing No. 200 sieve, direct shear test, miniature vane, and specific gravity. A brief description of data provided by each test is presented in Appendix H, pages H-1 to H-2. Table 4-3 presents the schedule of tests performed. Detailed information showing the matrix of samples tested, sample type, tests performed, and their locations is presented in Table 4-4.

4.2 RESULTS OF FIELD INVESTIGATIONS AND LABORATORY TESTING

Data from field investigations consists of CPT soundings, test pit, and boring logs. The CPT soundings include measurements of cone penetration resistance as well as shear-wave velocities. Laboratory geotechnical soil testing was performed according to the ASTM test methods, and results were provided by Teratest. Results from the field investigation and laboratory testing are discussed in more detail in the following sections.

4.2.1 Cone Penetration Test Soundings

During CPT operations, a standard cone is pushed down into the soil by a hydraulic ram. Data gathered from the CPTs include cone-tip resistance, local side friction, and pore pressures. The variation of the above parameters with depth was recorded. Results of the CPTs for Locations C-2-1 to C-2-15 were provided by HFA and are presented in Appendix B. Results from Locations C-2-16 to C-2-21 were provided by Gregg Drilling and Testing, Inc., and are also provided in Appendix B. The friction ratio (ratio of sleeve friction to cone-tip resistance), pore pressure ratio (ratio of pore pressure to cone-tip resistance), and soil behavior type are also provided in the figures.

In addition to standard CPT data collected, shear-wave velocities were measured at IR Site 2 area Locations C-2-6 and C-2-13 to a depth of 85 (from top of berm approximately 10 feet above site surface) and 84 feet below site surface, respectively. In the Additional Investigation Area between IR Sites 1 and 2, shear-wave velocities were measured at Location C-2-19 from the existing ground surface to a depth of approximately 95 feet bgs. Shear-wave velocity measurements are presented in Appendix B.

4.2.2 Test Pit Exploration Logs

Results of test pit explorations are summarized in Table 4-5. These explorations revealed soil cover thickness, types of soil, refuse items found, presence of odor and water, and any soil discoloration. In addition, 20-pound bulk samples were collected and sent to Teratest for testing. Index properties such as moisture and fines content were obtained.

The thickness of the existing soil covers varied from 2 inches to 2 feet over the refuse. In general, there were no liners observed, and less than 2 feet of soil cover existed in most areas. In

TABLE 4-3

SCHEDULE OF LABORATORY TESTS PERFORMED

Test	Method	Quantity
Atterberg Limits	ASTM D 4318	23
Moisture/Dry Density Analyses	ASTM D 2216/2937	48
Sieve & Hydrometer - Particle Size Analyses	ASTM D 422	21
Unconsolidated-Undrained Triaxial Shear	ASTM D 2850	5
Consolidated-Undrained Triaxial Shear	ASTM D 4767	5
Water Content	ASTM D 2216	1
Percent Passing No. 200	ASTM D 1140	30
Direct Shear	ASTM D 3080	4
Miniature Vane Shear	ASTM D 4648	25
Specific Gravity	ASTM D 854	3
Consolidation	ASTM D 2435	4

Notes:
ASTM - American Society for Testing and Materials

TABLE 4-4

SUMMARY OF LABORATORY TESTS PERFORMED

Sample ID	Sample Type	Laboratory Tests Performed											Location (Depth) (feet)
		Atterburg Limits	Moisture/Dry Density Analyses	Sieve & Hydrometer - Particle Size Analyses	Unconsolidated-Undrained	Consolidated-Undrained Triaxial Shear	Water Content	Percent Passing No. 200	Direct Shear	Miniature Vane Shear	Specific Gravity	Consolidation	
095-2-050	20# bag												TP-2-2 (3"-1.5')
095-2-051	20# bag												TP-2-1 (3"-1.0')
095-2-052	20# bag												TP-2-3 (3"-1.0')
095-2-053	20# bag												TP-2-4 (3"-2.0')
095-2-054	20# bag												TP-2-10 (3"-1.0')
095-2-055	20# bag												TP-2-8 (3"-1.5')
095-2-056	20# bag												TP-2-7 (3"-2.0')
095-2-057	3"x30" tube												B-2-12 (15'-17')
095-2-058	3"x30" tube												B-2-12 (25'-27')
095-2-059	2"x6" sleeve	X	X	X						X			B-2-12 (30'-31.5')
095-2-060	3"x30" tube												B-2-12 (40'-42')
095-2-061	2"x6" sleeve	X	X	X						X			B-2-12 (45'-46.5')
095-2-062	2"x6" sleeve		X										B-2-7 (5'-6.5')
095-2-063	2"x6" sleeve												B-2-7 (10'-11.5')
095-2-064	2"x6" sleeve		X										B-2-7 (15'-16.5')
095-2-065	3"x30" tube												B-2-12 (50'-52')
095-2-066	2"x6" sleeve	X	X	X						X			B-2-12 (55'-56.5')
095-2-067	3"x30" tube	X	X	X						X			B-2-12 (72'-74')
095-2-068	2"x6" sleeve												B-2-12 (75'-76.5')
095-2-069	2"x6" sleeve		X					X					B-2-8 (5'-6.5')
095-2-070	2"x6" sleeve		X					X					B-2-8 (10'-11.5')
095-2-071	2"x6" sleeve		X					X					B-2-8 (15'-16.5')
095-2-072	2"x6" sleeve		X					X					B-2-8 (20'-21.5')
095-2-073	2"x6" sleeve												B-2-8 (25'-26.5')
095-2-074	2"x6" sleeve		X					X					B-2-8 (30'-31.5')
095-2-075	3"x30" tube	X	X	X						X			B-2-8 (35'-37')
095-2-076	bag							X					B-2-8 (45'-46.5')
095-2-077	bag							X					B-2-8 (50'-51.5')
095-2-078	bag												B-2-8 (55'-56.5')
095-2-079	2"x6" sleeve		X					X					B-2-8 (60'-61.5')
095-2-080	2"x6" sleeve												B-2-8 (65'-66.5')
095-2-081	2"x6" sleeve		X					X					B-2-8 (70'-71.5')
095-2-082	3"x30" tube		X					X					B-2-9 (5'-7')
095-2-083	2"x6" sleeve		X					X					B-2-9 (10'-11.5')
095-2-084	2"x6" sleeve		X					X	X				B-2-9 (15'-16.5')
095-2-085	3"x30" tube		X					X					B-2-9 (20'-22')
095-2-086	3"x30" tube		X					X					B-2-9 (25'-27')
095-2-087	3"x30" tube												B-2-9 (30'-32')
095-2-088	2"x6" sleeve		X					X					B-2-9 (35'-36.5')
095-2-089	3"x30" tube												B-2-9 (40'-42')
095-2-090	2"x6" sleeve		X					X					B-2-9 (45'-46.5')
095-2-091	2"x6" sleeve												B-2-9 (50'-51.5')
095-2-092	bag							X					B-2-9 (55'-56.5')

TABLE 4-4

SUMMARY OF LABORATORY TESTS PERFORMED

Sample ID	Sample Type	Laboratory Tests Performed											Location (Depth) (feet)
		Atterburg Limits	Moisture/Dry Density Analyses	Sieve & Hydrometer - Particle Size Analyses	Unconsolidated-Undrained	Consolidated-Undrained Triaxial Shear	Water Content	Percent Passing No. 200	Direct Shear	Miniature Vane Shear	Specific Gravity	Consolidation	
095-2-093	bag												B-2-9 (60'-61.5')
095-2-094	bag							X					B-2-9 (65'-66.5')
095-2-095	bag												B-2-9 (70'-71.5')
095-2-096	2"x6" sleeve												B-2-10 (5'-6.5')
095-2-097	2"x6" sleeve												B-2-10 (10'-11.5')
095-2-098	3"x30" tube	X	X	X						X			B-2-10 (15'-17')
095-2-099	2"x6" sleeve		X					X					B-2-10 (20'-21.5')
095-2-100	2"x6" sleeve							X	X				B-2-10 (25'-26.5')
095-2-101	2"x6" sleeve		X					X					B-2-10 (30'-31.5')
095-2-102	3"x30" tube												B-2-10 (35'-37')
095-2-103	3"x30" tube		X					X					B-2-10 (40'-42')
095-2-104	3"x30" tube												B-2-10 (45'-47')
095-2-105	2"x6" sleeve	X	X	X						X			B-2-10 (50'-51.5')
095-2-106	3"x30" tube												B-2-10 (55'-57')
095-2-107	bag							X					B-2-10 (60'-61.5')
095-2-108	bag												B-2-10 (65'-66.5')
095-2-109	2"x6" sleeve							X					B-2-10 (75'-76.5')
095-2-110	2"x6" sleeve							X	X				B-2-11 (5'-6.5')
095-2-111	3"x30" tube												B-2-11 (10'-12')
095-2-112	2"x6" sleeve		X					X					B-2-11 (15'-16.5')
095-2-113	bag												B-2-11 (20'-21.5')
095-2-114	2"x6" sleeve		X					X					B-2-11 (25'-26.5')
095-2-115	3"x30" tube												B-2-11 (30'-32')
095-2-116	3"x30" tube	X	X	X						X			B-2-11 (35'-37')
095-2-117	3"x30" tube												B-2-11 (40'-42')
095-2-118	3"x30" tube	X	X	X						X			B-2-11 (45'-47')
095-2-119	3"x30" tube	X	X	X						X			B-2-11 (55'-57')
095-2-120	3"x30" tube	X	X	X						X			B-2-11 (70'-72')
095-2-121	bag												B-2-11 (75'-76.5')
095-2-122	bag							X					B-2-11 (80'-81.5')
095-2-123	2"x6" sleeve												B-2-11 (90'-91.5')
095-2-124	bag												B-2-11 (105'-106.5')
095-2-125	3"x30" tube	X	X	X						X			B-2-11 (110'-112')
095-2-126	3"x30" tube	X	X	X						X			B-2-11 (120'-122')
095-2-127	bag												B-2-11 (130'-132')
095-2-128	bag												B-2-7A (5'-6.5')
095-2-129	bag												B-2-7A (10'-11.5')
095-2-130	bag												B-2-7A (15'-16.5')
095-2-131	3"x30" tube	X	X	X						X			B-2-7A (25'-27')
095-2-132	2"x6" sleeve	X	X	X						X			B-2-7A (30'-31.5')
095-2-133	3"x30" tube	X	X	X						X			B-2-7A (35'-37')
095-2-134	2"x6" sleeve							X	X				B-2-7A (40'-41.5')
095-2-135	bag												B-2-7A (45'-46.5')
095-2-136	bag							X					B-2-7A (50'-51.5')

TABLE 4-4

SUMMARY OF LABORATORY TESTS PERFORMED

Sample ID	Sample Type	Laboratory Tests Performed											Location (Depth) (feet)
		Atterburg Limits	Moisture/Dry Density Analyses	Sieve & Hydrometer - Particle Size Analyses	Unconsolidated-Undrained	Consolidated-Undrained Triaxial Shear	Water Content	Percent Passing No. 200	Direct Shear	Miniature Vane Shear	Specific Gravity	Consolidation	
095-2-137	2"x6" sleeve												B-2-15 (5.5'-6')
095-2-138	bag												B-2-15 (10'-11.5')
095-2-139	3"x30" tube												B-2-15 (15'-17')
095-2-140	2"x6" sleeve												B-2-15 (20.5'-21')
095-2-141	3"x30" tube	X	X			X				X	X	X	B-2-15 (30'-32.5')
095-2-142	3"x30" tube		X	X		X				X			B-2-15 (40'-42.5')
095-2-143	2"x6" sleeve												B-2-15 (50'-50.5')
095-2-144	bag												B-2-15 (60'-60.5')
095-2-145	bag												B-2-15 (70'-71.5')
095-2-146	bag												B-2-15 (80'-81.5')
095-2-147	3"x30" tube	X	X		X							X	B-2-15 (88'-90.5')
095-2-148	3"x30" tube		X										B-2-15 (111'-113.5')
095-2-149	3"x30" tube												B-2-15 (161'-163')
095-2-150	bag												B-2-15 (180'-181.5')
095-2-151	bag												B-2-15 (191'-192.5')
095-2-152	2"x6" sleeve												B-2-14 (5.5'-6')
095-2-153	bag												B-2-14 (10'-11.5')
095-2-154	3"x30" tube												B-2-14 (15'-17.5')
095-2-155	2"x6" sleeve												B-2-14 (19'-19.5')
095-2-156	3"x30" tube	X	X	X		X				X	X	X	B-2-14 (30'-32.5')
095-2-157	3"x30" tube		X			X				X			B-2-14 (40'-42.5')
095-2-158	2"x6" sleeve												B-2-14 (50'-51.5')
095-2-159	bag												B-2-14 (60'-61.5')
095-2-160	bag												B-2-14 (70'-71.5')
095-2-161	bag												B-2-14 (80'-80.8')
095-2-162	3"x30" tube	X	X	X	X								B-2-14 (90'-92.5')
095-2-163	3"x30" tube		X										B-2-14 (100'-102.5')
095-2-164	3"x30" tube				X								B-2-14 (120'-122.25')
095-2-165	3"x30" tube				X								B-2-14 (150'-152.5')
095-2-166	2"x6" sleeve	X	X	X						X			B-2-13 (25.5'-26')
095-2-167	3"x30" tube	X	X	X		X				X	X	X	B-2-13 (40'-42.5')
095-2-168	2"x6" sleeve		X										B-2-13 (70.5'-71')
095-2-169	bag												B-2-13 (80'-81.5')
095-2-170	3"x30" tube	X	X		X					X			B-2-13 (90'-92.5')
095-1-171	3"x30" tube						X						B-2-13 (100'-102.5'

Notes:
see Appendix H for confining pressures associated with triaxial shear testing

TABLE 4-5

SUMMARY OF TEST PIT EXPLORATIONS

Test Pit #	Total Depth Excavated (feet)	Depth to Refuse (feet)	Visual Soil Classification of Existing Cover Soil with Munsell Colors Description and Other Geomaterials Found	Items Found	Comments
TP-2-1	1.50	1.00	3- to 4-inch grass/root and soil cover above SP-moist fine sand with <10 percent LPF, olive brown 2.5 yellow red (4/3), slight moisture with shell fragments	Minor metal and plastic in a fine sand soil matrix	Soil and refuse discoloration-reddish/orange color
TP-2-2	2.50	1.50	3-inch grass/root and soil cover above SP-moist fine sand with <10 percent LPF, olive brown 2.5 yellow red (4/3), slight moisture with shell fragments	Paper, plastic wood, etc.	No odor
TP-2-3	2.00	1.00	3-inch grass/root and soil cover above SP-moist fine sand with <10 percent LPF, olive brown 2.5 yellow red (4/3), slight moisture with shell fragments	Rubber and fire hose pieces, plastic, etc.	Refuse in a sandy silt matrix, slight moisture and no odors
TP-2-4	2.50	2.00	6-inch grass/root and soil cover above SM-sandy silt, very dark grayish brown 2.5 yellow red (4/2), medium plasticity and consolidated	Plastic and > 20 percent paper	Refuse in a sandy silt matrix, slight moisture and no odors
TP-2-5	2.00	0.16	2-inch grass/root and soil cover above sandy silt mixed with construction debris	Construction debris consisting of concrete, pipe, gravel, some brick, etc.	No sample collected due to approximately 20 percent of construction debris
TP-2-6	1.00	2.00	Less than 2-inch grass/root and soil cover above SP-moist fine sand with <10 percent LPF, olive brown 2.5 years (4/3), slight moisture with shell fragments	Stained metal, wood, paper, etc.	Refuse discoloration-reddish/orange color, soil matrix dark brown silty sand
TP-2-7	4.00	0.25	3-inch grass/root and soil cover above (1) SM-sandy silt, very dark grayish brown 2.5 yellow red (4/2), medium plasticity and consolidated with approximately 5 percent construction debris. (2) SP-moist fine sand with <10 percent LPF, olive brown 2.5 yellow red (4/3), slight moisture with shell fragment, extends from approximately 2 to 3 feet	Construction debris consisting of asphalt, brick, stone etc. At 3 feet, paper, plastic, wire, wood, etc.	Based on the presence of 5 percent construction debris from 3 inches to 3 feet, would not consider suitable soil cover, refuse at 3 feet has discoloration-reddish/orange color
TP-2-8	2.50	1.50	3-inch grass/root and soil cover above SP-moist fine sand with <10 percent LPF, olive brown 2.5 yellow red (4/3), slight moisture with shell fragments	Wood, metal	Refuse discoloration-reddish/orange color, soil matrix dark brown silty sand and no odors

TABLE 4-5

SUMMARY OF TEST PIT EXPLORATIONS

Test Pit #	Total Depth Excavated (feet)	Depth to Refuse (feet)	Visual Soil Classification of Existing Cover Soil with Munsell Colors Description and Other Geomaterials Found	Items Found	Comments
TP-2-9	3.00	0.16	2-inch grass/root and soil cover above (1) SM-sandy silt, very dark grayish brown 2.5 yellow red (4/2), medium plasticity and consolidated with approximately > 10 percent construction debris, typical refuse encountered at 1.5 feet below ground surface (2) SM-moist fine sand with silt, olive brown 2.5 yellow red (4/3), slight moisture with shell fragment, extends from approximately 1 to 1.5 feet	Construction debris consisting of asphalt, brick, stone etc. At 1.5-feet paper, plastic, wire, wood, etc.	Based on the presence of 10 percent construction debris from 2 inches to 1 foot would not consider suitable soil cover, refuse at 1.5 feet has discoloration-reddish/orange color
TP-2-10	2.50	1.00	3-inch grass/root and soil cover above SM-moist fine sand with silt, medium plasticity, olive brown 2.5 yellow red (4/3)	Wood, metal, cloth, paper (20 percent), etc.	Refuse in a sandy silt matrix, test pit excavated to 2.5 feet and water entered excavation and stabilized at 2 feet below ground surface
TP-2-11	3.00	0.16	2-inch grass/root and soil cover above (1) SM-sandy silt, very dark grayish brown 2.5 yellow red (4/2), medium plasticity and consolidated with approximately > 10 percent construction debris, typical refuse encountered at 1.5 feet below ground surface	Construction debris consisting of asphalt, brick, stone etc., refuse mostly wood	Based on the presence of 10 percent construction debris from 2 inches to 1 foot would not consider suitable soil cover. refuse at 1.5 feet has discoloration-reddish/orange color
TP-2-12	3.00	2.00	6-inch grass/root and soil cover above SM-moist fine sand with silt, medium plasticity, olive brown 2.5 yellow red (4/3)	Glass, plastic, paper (10 percent), etc.	Refuse in a sandy silt matrix, test pit excavated to 3 feet and water encountered

Notes:
LPF - low plasticity fines
SM - silty sand
SP - poorly graded sand

Test Pits TP-2-5, TP-2-7, TP-2-9, and TP-2-11, construction debris in a silty soil matrix was encountered less than 6 inches below the grass cover. Concrete, asphalt, brick, and some pipe were present and are assumed to be associated with the construction of taxiways and runways nearby. The soil layer containing the construction material was either graded directly into the refuse layer or in the case of Test Pits TP-2-7 and TP-2-9, a thin layer of clean silty sand material separated the unit from the underlying refuse.

For the most part, cover soil consisted of fine-grained materials. Poorly graded sand (SP) was observed in Test Pits TP-2-1, TP-2-2, TP-2-3, TP-2-6, and TP-2-8. Silty sand (SM) was found in Test Pits TP-2-4, TP-2-7, and TP-2-10.

Refuse was observed in all of the test pits explored. Waste material included items such as wire, asphalt, miscellaneous wood, plastic, hoses, and metal objects. No significant odor was detected while sampling in any of the test pits. However, soil discoloration was observed at Test Pits TP-2-1, TP-2-6, TP-2-7, TP-2-8, TP-2-9, and TP-2-11.

Most soil samples collected were moist due to rainy weather conditions and proximity to the bay. Significant water flow (seepage) was observed while excavating in Test Pits TP-2-10 and TP-2-12 at 2.5 to 3.0 feet bgs. The observations of seepage and proximity of the site to the bay indicate a high water table for the site.

The test pit logs for all 12 test pits are included in Appendix J. Test pit locations are shown in Figure 2-1.

4.2.3 Soil Boring Logs

Data collected from soil borings are summarized in boring logs presented in Appendix C. Metal avoidance clearance logs showing that the boreholes are safe to drill are also presented in Appendix C. The borings provide information on geotechnical characteristics of subsurface soils, observed soil type, stratigraphic boundaries, consistency/strength properties (blow counts), and any odor, water, or soil discolorations present. Selected samples taken from soil borings are tested to evaluate index and engineering properties used to perform geotechnical and seismic analyses.

Four geological units that were identified through the soil borings included fill material, fine-grained sensitive Young Bay Mud, dense sands, and stiff, silty clay material.

Upland soil borings indicate that fill material consisted mostly of coarse grained soils [SM, clayey sand (SC), SP-SM, SP] and extended from the ground surface to a maximum depth of about 45 feet bgs. The fill is thicker to the south and to the west. The fill material was encountered throughout the site and contained gravel, shell fragments, wood chips, and concrete rubble. In general, the fill was poorly compacted with blow counts mostly in the low single digits.

Fine-grained, cohesive soil commonly known as Young Bay Mud was observed below the fill material. This geological unit extended up to 70 feet bgs. Blow counts recorded in clay layers were relatively low. Below the Young Bay Mud, a light olive brown sand layer with significant fines was observed. This layer comprises the geologic units known as Bay Sediments and Merritt Sand and is relatively dense with blow counts consistently reaching 50 (refusal).

Soils observed from upland borings were generally moist or saturated due to the location and history of the site. No piezometer readings were performed.

Offshore soil borings (B-2-1 through B-2-6) were conducted and reached a depth of approximately 60 feet below the mud line. Boring locations are shown in Figure 2-1. Soil Borings B-2-1 to B-2-3, drilled on the south side of IR Site 2, showed that mostly sandy soils (SM) are present from the existing mudline to the maximum explored depth of 60 feet. Soil Borings B-2-4 through B-2-6, drilled west of IR Site 2, encountered Young Bay Mud sediments from the existing mudline to depths ranging from 30 to 75 feet. The Young Bay Mud is underlain by the dense Merrit Sand layer.

4.2.4 Laboratory Test Results

The final report from Teratest, including COCs, summary of test results, and individual laboratory data sheets, are provided in Appendix H.

The following classification and index soil property testing was conducted:

- Moisture content and dry unit weight of soils
- Results of percent passing No. 200 sieve
- Specific gravity test results
- Atterberg Limits test results (liquid limit, plastic limit, and plasticity index) summarized on plasticity charts
- Particle-size distribution tests
- Consolidation tests on clayey soils

In situ shear strength properties of selected specimens of relatively undisturbed soils were evaluated based on direct shear, UU triaxial, and CU triaxial tests as follows:

- The results of three direct shear tests conducted on SM
- Shear strength testing of fine-grained soils included four UU triaxial tests and four sets of CU triaxial tests with pore pressure measurements
- Results of CU triaxial tests (including three consolidation rate readings)

Laboratory miniature vane shear tests (ASTM Test Method D 4648) were performed on Bay Sediments (Young Bay Mud or loose sandy soils) classified in the field as "soft or loose" to obtain estimates of their undrained shear strength. This test method was selected to minimize sample disturbance of soft/loose sediments since it is performed directly on the sample without the need of extruding the material rings or Shelby tubes. Results of miniature vane shear tests are presented in Appendix H.

4.3 GEOLOGIC FEATURES

The Alameda Point site area is located on Alameda Island on the eastern side of the central San Francisco Bay. The San Francisco Bay region is located within an elongated basin or valley that extends southeasterly to the Santa Clara Valley. The San Francisco Bay and Santa Clara Valley are bounded by the Santa Cruz Mountains to the southwest and the East Bay Hills and Diablo Range to the northeast.

4.3.1 Physiography

The project area is located in the Coast Ranges geologic/geomorphic province of central and northern California. The Coast Ranges extend from the Transverse Ranges province approximately 500 kilometers south of the project site to about 400 kilometers north where the province meets the Klamath Mountains. The Coast Ranges province is bordered on the west by the Pacific Ocean and to the east by the Great Valley province. The Coast Ranges have a general northwest-southeast orientation and are characterized by northwest-southeast trending folds and faults.

The water depth in the eastern part of the bay, where the site is located, is generally very shallow, and at low tide, the muddy bay floor is visible through the shallow water. Onshore, the eastern margin of the bay is generally a very flat, low-lying plain about 2 to 6 miles wide, underlain by fills and tidal marshes. In the natural environment, the channel north of Alameda Point was a natural stream channel, called San Antonio Creek, which flowed from Lake Merritt. Both of these features were under the influence of tides.

Alameda Island is a low-lying flat area composed partly of artificial fill and partly of tidal-flat marshy sediments (Figure 4-1). Originally, the island was a peninsula connected to land on the southeast, but dredging in the late 19th and early part of the 20th century deepened and extended the San Antonio Creek channel southeasterly to form Oakland Inner Harbor, thus creating the island. The island was enlarged by extending it northwesterly with materials dredged from the surrounding bay. IR Site 2, the project area, is located entirely over, and composed of, this dredged material.

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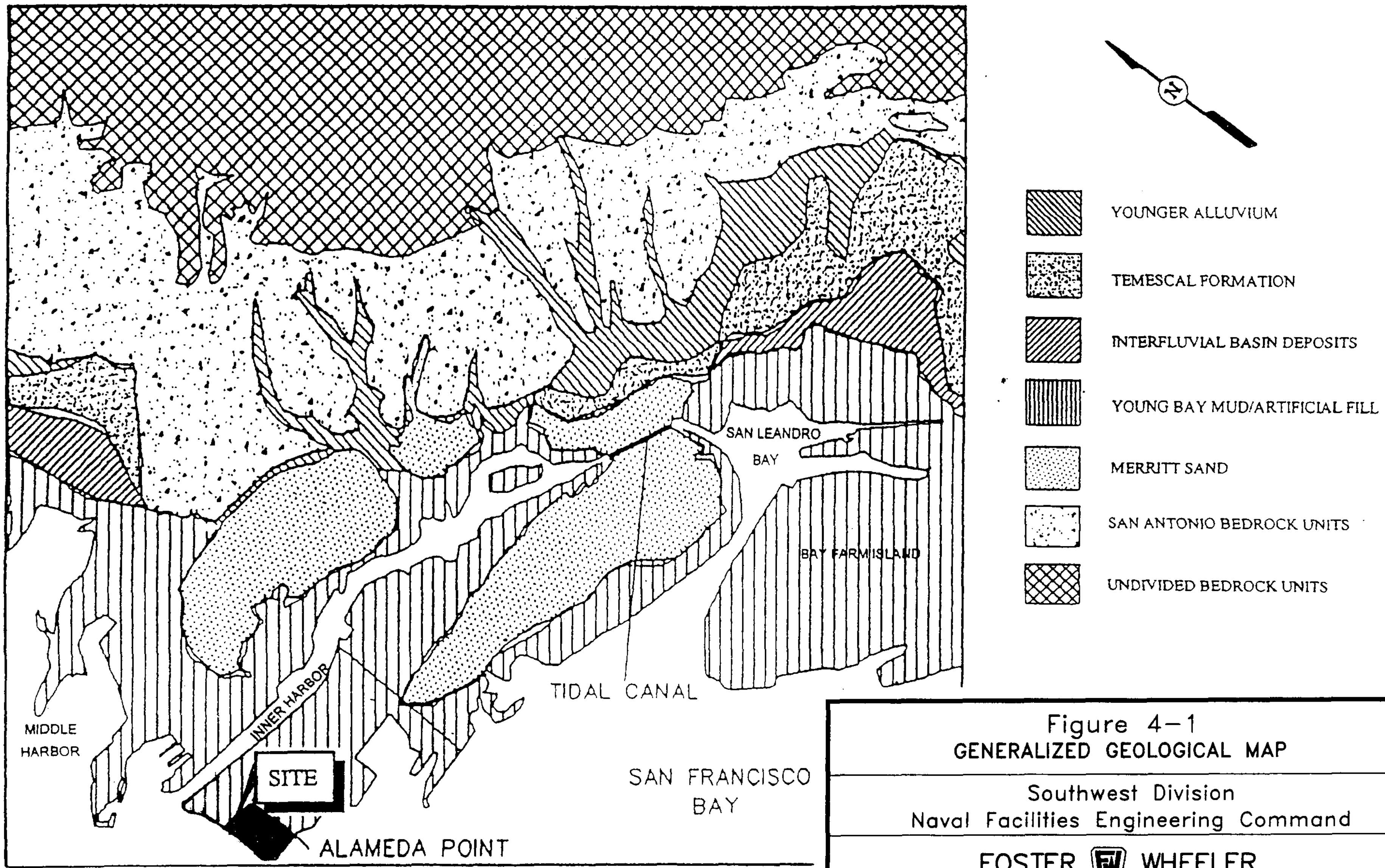

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-1
GENERALIZED GEOLOGICAL MAP

Southwest Division
Naval Facilities Engineering Command

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4.3.2 Stratigraphy

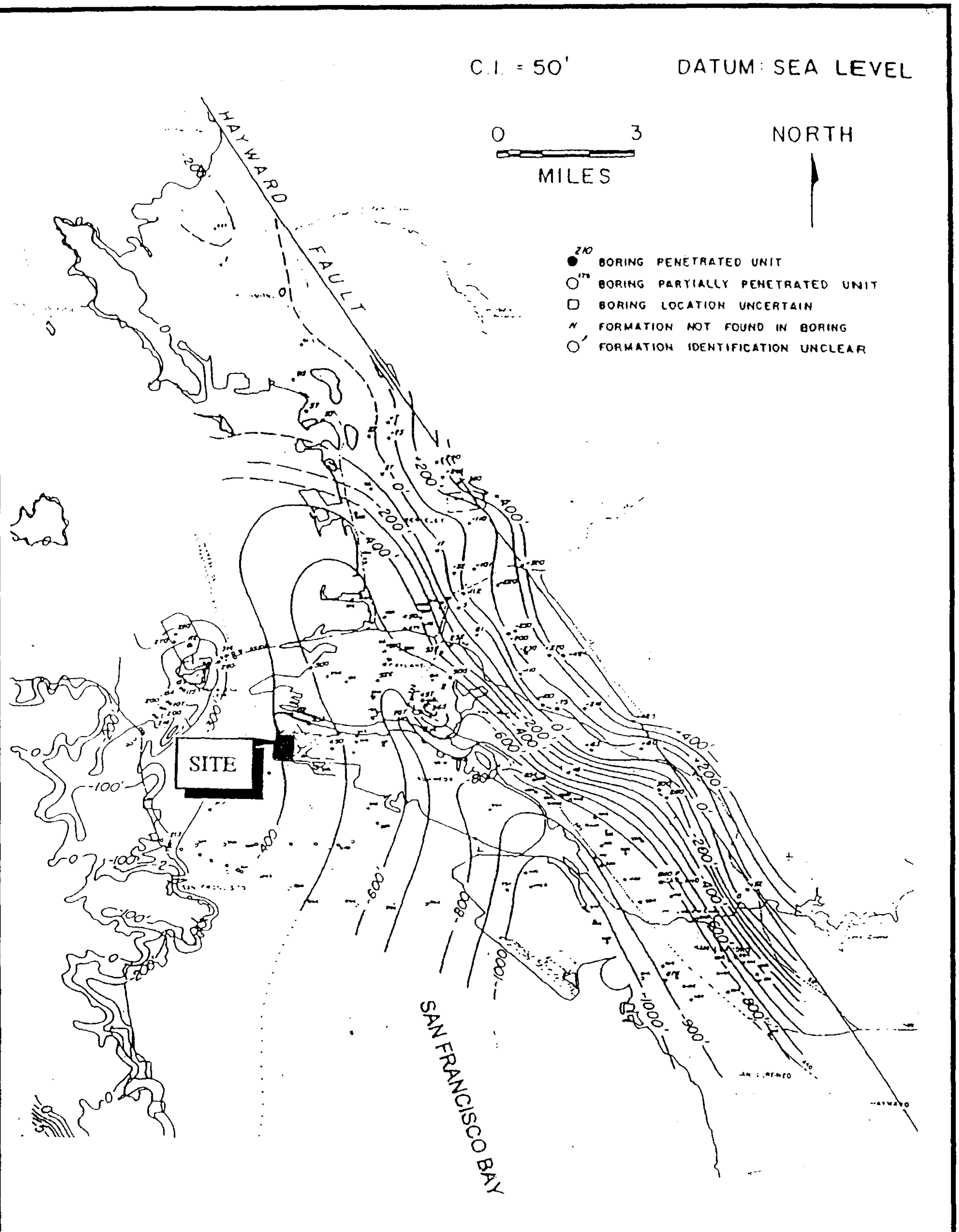
Rocks of the Coast Ranges province consist of sedimentary, metamorphic, volcanic, and igneous ranging in age predominantly from the Mesozoic (Jurassic/Cretaceous) to recent (Holocene). The Franciscan Complex underlies most of the San Francisco Bay area and consists of sedimentary, metamorphic, volcanic, and igneous rocks. These rocks are believed to have accreted onto the North American plate during subduction events that ended in the Miocene time (Page, 1992). Parts of the accreted assemblage form coherent, solid rock, whereas other parts of the complex have been sheared and disrupted, and consist a melange of exotic blocks of basalt, chert, limestone, gabbro, blueschist, eclogite, and amphibolite that are embedded in a tectonic paste of sheared shale, graywacke sandstone, or serpentinite (Wahrhaftig, 1989; Page, 1992).

The Great Valley Sequence, which underlies much of the East Bay Hills east of the site, consists of a late Jurassic to Cretaceous-age assemblage of marine sandstone, shales, and conglomerates. The sequence is generally much more coherent and regular than the Franciscan Complex and possesses greater stratal continuity and lacks melanges (Page, 1992). The Great Valley Sequence is up to 14,000 feet thick and was likely deposited in a marine basin between the subduction/accretionary prism, where the Franciscan Formation was formed. A volcanic arc was located about where the western slope of the Sierra Nevada Mountains are currently located (Wahrhaftig, 1989).

The Franciscan Complex and Great Valley Sequence form the "basement rocks" throughout most of the Bay Area, including the site. Deposited onto these basement rocks are Quaternary sediments and Tertiary-age marine and non-marine sedimentary rocks (for example, the Contra Costa Group and Santa Clara Formations). The younger sediments, rocks, and the surrounding basement were all uplifted by folding and faulting in a relatively recent geologic time indicating that tectonic deformation of the area is still active and ongoing.

The site area is underlain by the Franciscan Complex basement. This basement rock is at a depth of about 400 to 500 feet below the site (Figure 4-2). These basement rocks are overlain by a sequence of non-indurated sediments deposited primarily during Quaternary time (for example, the past million years). These sedimentary units record a sequence of fillings and evacuations of San Francisco Bay in response to global glacial/climate changes and local tectonics. One of the latest stream system adjustments in the region is the San Antonio-Glen Echo-Trestle Glen Creek system, which joins at Lake Merritt and has collectively cut a new channel through the Merritt Sand called San Antonio Slough. This present-day channel lies approximately 2,000 feet north of a late-Wisconsin-age (more than 11,000 years old) channel, which formerly flowed directly beneath the central part of what is now Alameda Point (Rogers and Figuers, 1991).

Figure 4-3 summarizes the stratigraphy at the site and surrounding region and shows that the sediments are categorized into five to nine geologic units or formations. These formations are briefly described below starting from youngest (fill) to oldest (Alameda Formation).



SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-2
STRUCTURE CONTOUR MAP ON TOP OF FRANCISCAN FORMATION (BASEMENT)

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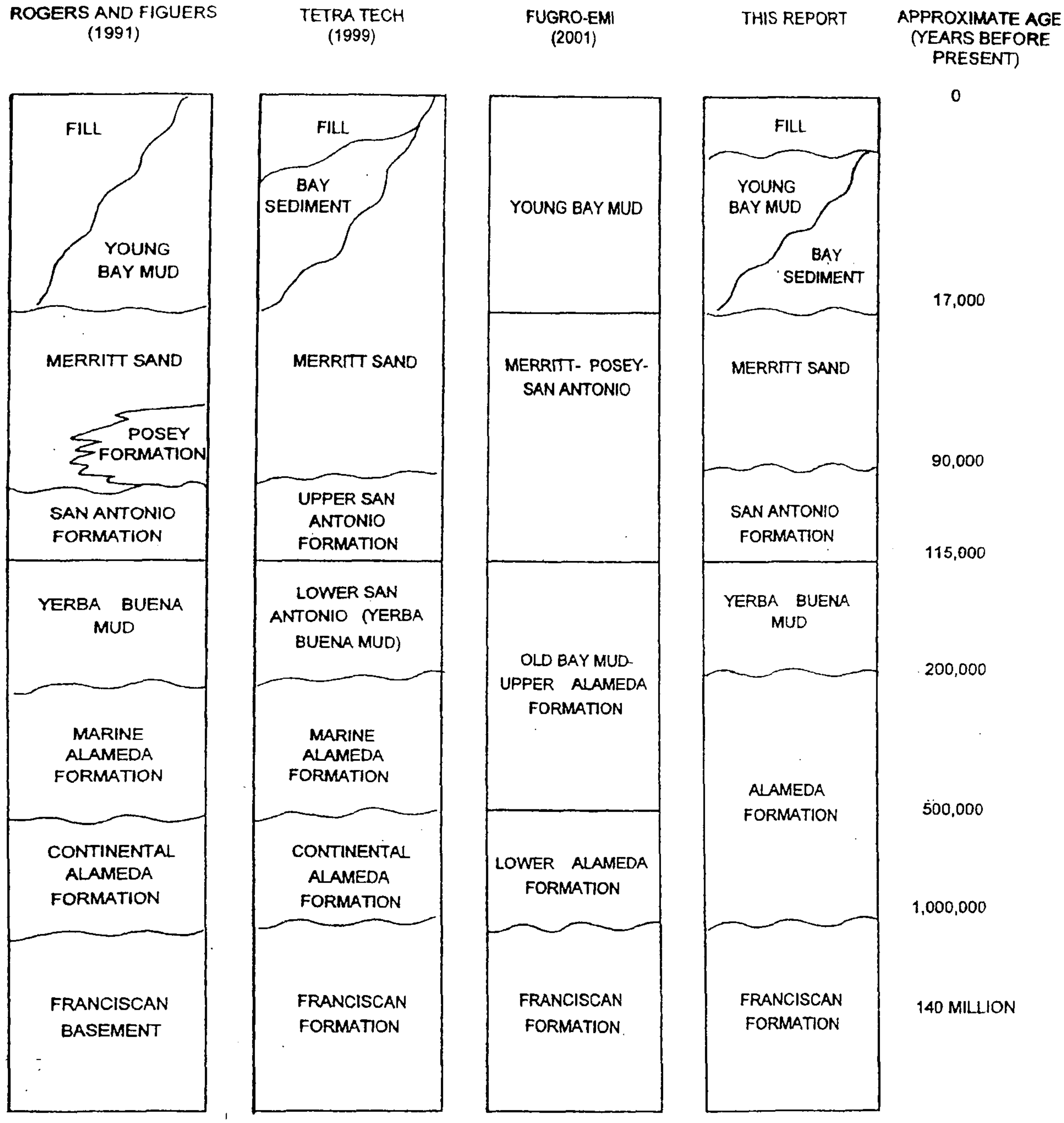

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-3
STATIGRAPHIC COLUMN

Southwest Division
Naval Facilities Engineering Command

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Figures 4-4 and 4-5 are geologic cross sections showing the subsurface distribution of the various formations. The geotechnical properties of these formations are discussed in more detail in Section 4.4.1. It should be noted that the geotechnical characterization may differ slightly from the geological characterization because the geotechnical characterization is based more on engineering properties, whereas the geological characterization includes factors such as depositional environment and age. In the discussions that follow, numerous references are made to the investigations performed for the proposed replacement of the east span of the San Francisco-Oakland Bay Bridge (SFOBB). The comprehensive investigation (Fugro-EMI, 1999) conducted for the SFOBB is relevant to this current Remedial Investigation (RI) because of the SFOBB's proximity (only 1.5 miles away) and nearly identical geological/geotechnical conditions to Alameda Point.

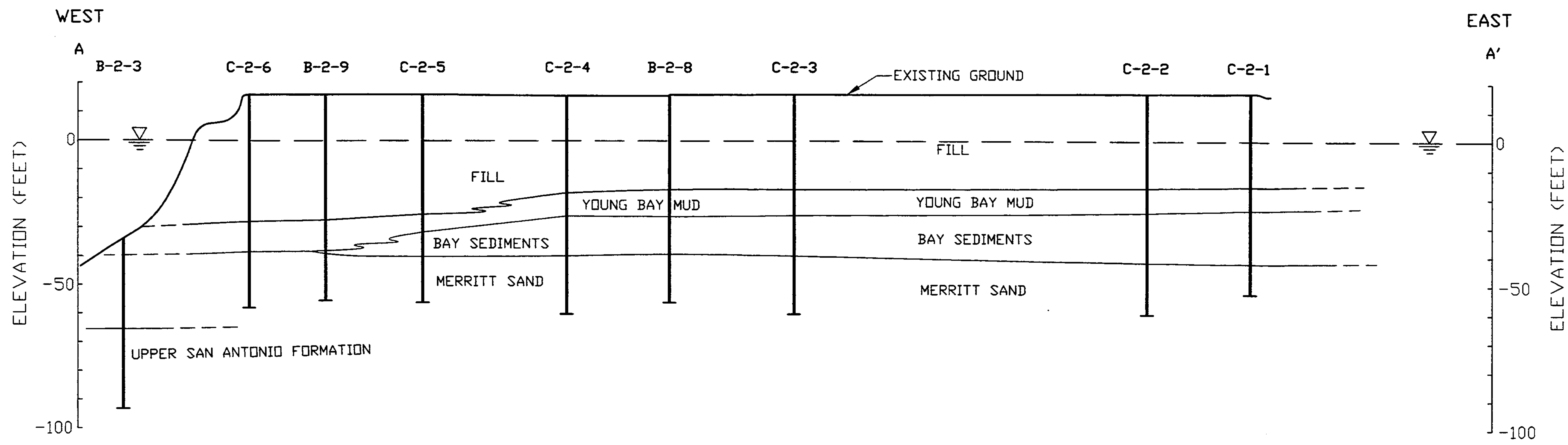
4.3.2.1 Fill

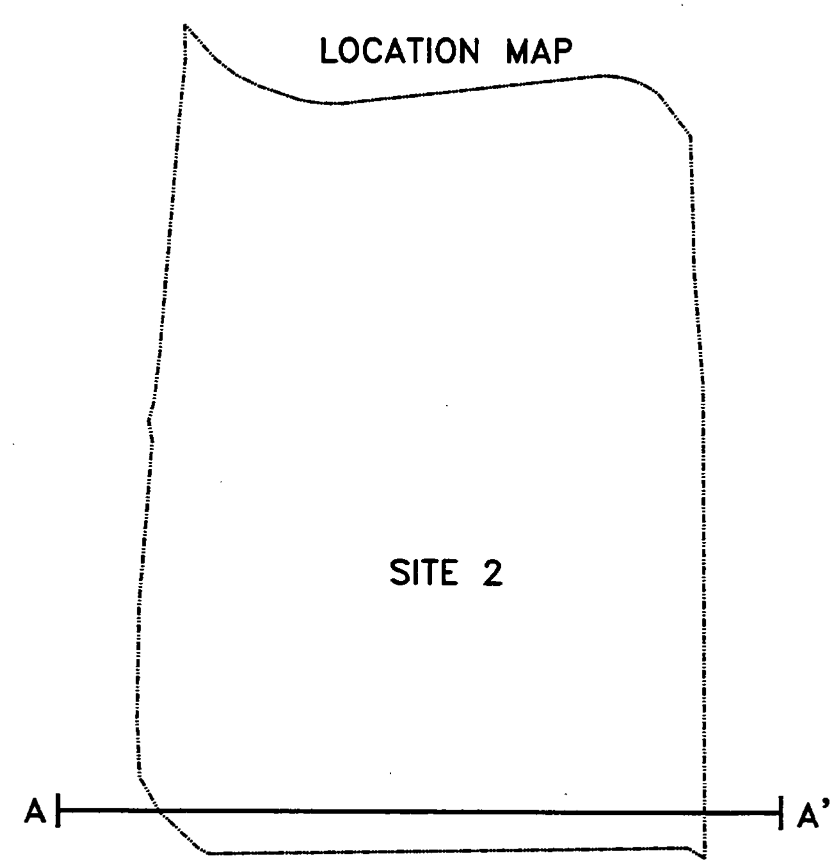
The fill encountered at most of the site is composed of mixtures of sand, silt, and clay dredged from the surrounding bay and a rock dike to retain the fill in place. The fill ranges in thickness from about 25 feet in the northwest to 45 feet in the southwest part of IR Site 2. The varying thickness is a result of natural variation in the depth of the estuary before filling, which began in the late 1800s. The Merritt Sand served as one of the primary sources of the fill. The fill typically has abundant shell fragments and debris including gravel. The strength of the fill varies widely because of the wide variety of materials within.

The existing waste material in the fill is not well-defined due to lack of sufficient information on the waste disposal history at the site. A description of the possible types of waste material stored at the site is provided in the Operable Unit (OU)-3 RI Report prepared by Tetra Tech EM, Inc. (TtEMI) (TtEMI, 1999) and in Sections 1.1.2 and 4.2.2 of this report. Also, the existence of ordnance and explosives waste (OEW) at the site has been a major concern and a critical part of the investigation and remediation activities conducted by FWENC. Section 3.7 discusses the OEW found from the surface sweep and Time-Critical Removal Action (TCRA).

4.3.2.2 Young Bay Mud

The Young Bay Mud is of Holocene-latest Pleistocene age (less than about 15,000 years old) and is the youngest naturally occurring unit in the site area. Although commonly referred to as mud, the unit contains mixtures of silts and fine-grained sand. The material was deposited within the bay and the surrounding estuaries and tidal flats. The unit is generally very dark gray with marine shells and organic materials. The unit is generally soft, but can be firm locally. The shear-wave velocity of the Young Bay Mud measured at the SFOBB was generally in the 400 to 650 feet per second (ft/sec) range. Measurement of the velocity of seismic waves, such as shear-wave velocity, can provide an indication of the density and firmness (hardness) of soils and rocks. For example, the National Earthquake Hazard Reduction Program (NEHRP) guidelines [Federal Emergency Management Agency 273 (FEMA 273), 1997] provide site class definitions to





LEGEND

B-X-X	BOREHOLE
C-X-X	CPT LOCATION
———	SITE OUTLINE

200 100 0 200 400
SCALE IN FEET
VERTICAL EXAGGERATION 5:1

SOURCE:HUSHMAND ASSOCIATES, INC.

Figure 4–4
GEOLOGIC CROSS SECTION A–A'

SOUTHWEST DIVISION
NAVAL FACILITIES ENGINEERING COMMAND

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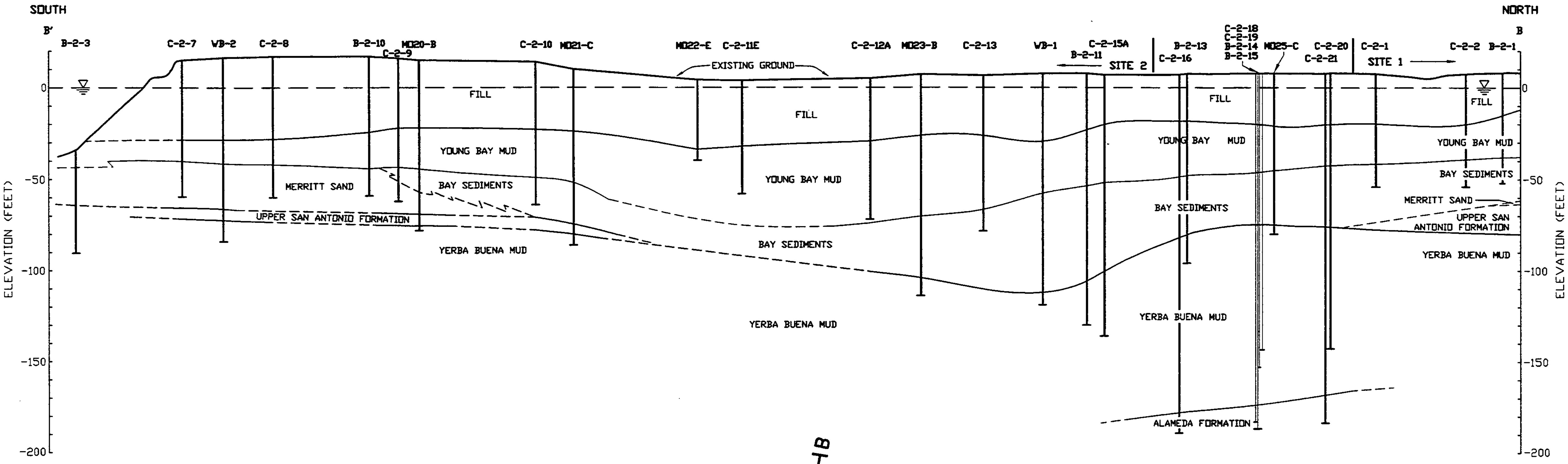

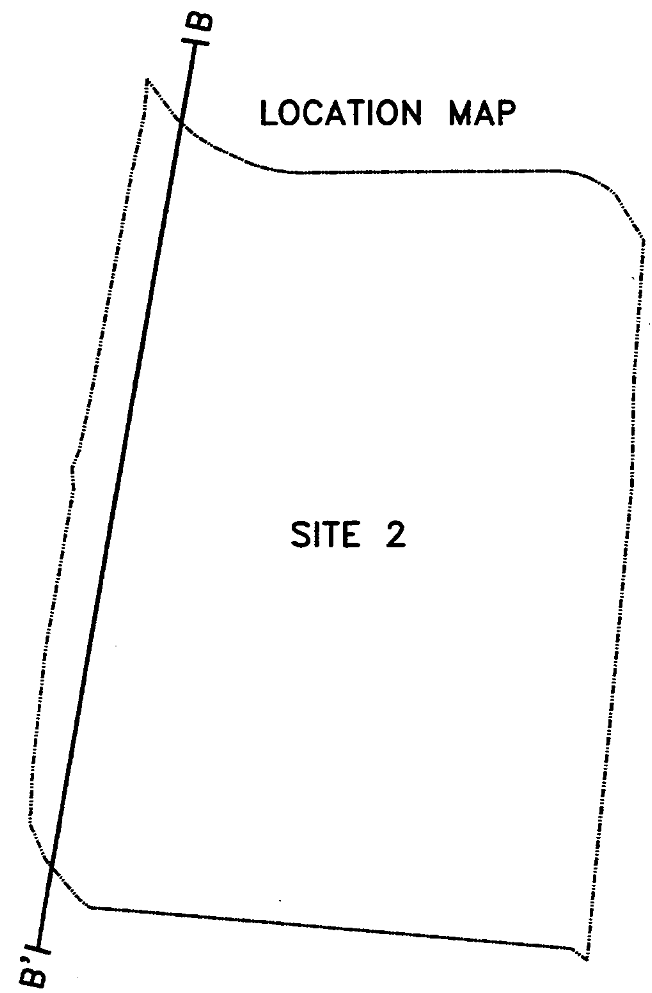

LEGEND

B-X-X	BOREHOLE
C-X-X	CPT LOCATION
————	SITE OUTLINE

300 150 0 300 600

SCALE IN FEET

VERTICAL EXAGGERATION 5:1

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4–5

GEOLOGIC CROSS SECTION B–B'

SOUTHWEST DIVISION
NAVAL FACILITIES ENGINEERING COMMAND

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include effects of the site local geology on estimated site design ground motions according to soil type and average shear-wave velocity of the top 100 feet of the site soils. The following is a summary of NEHRP site classifications based on soil type and shear-wave velocity:

- **Class A:** Hard rock with measured shear-wave velocity (Vs) > 5000 ft/sec
- **Class B:** Rock with 2,500 ft/sec < Vs < 5,000 ft/sec
- **Class C:** Very dense soil and soft rock with 1,200 ft/sec < Vs ≤ 2,500 ft/sec
- **Class D:** Stiff or dense soil with 600 ft/sec < Vs ≤ 1,200 ft/sec
- **Class E:** Any profile with more than 10 feet of soft clay or a soil profile with Vs < 600 ft/sec
- **Class F:** Soils requiring site-specific evaluations such as peats and/or highly organic clays, soils vulnerable to potential failure or collapse, very high plasticity clays, and very thick soft/medium stiff clays

Another example of correlating shear-wave velocity with soil type and density is provided by Tinsley and Fumal (1985) in Table 13 of their publication. The table summarizes correlations among relative bulk density, penetration resistance, shear-wave velocity, and calculated impedance values of surficial geologic units. Therefore, based on the NEHRP definitions above, the Young Bay Mud is classified as soft clay.

The Young Bay Mud is thinnest in the eastern and southern parts of IR Site 2 and is thickest in the northern part of IR Site 2 where it appears to represent an ancient channel fill. The maximum thickness in the axial part of the channel is about 45 to 50 feet. Where the unit is still accumulating in the deeper parts of the bay, the unit is as thick as approximately 100 feet (Trask and Rolston, 1951). In the vicinity of the SFOBB, just north of Alameda Point, the unit is about 70 to 75 feet thick.

In previous reports (TtEMI, 1999), the Young Bay Mud unit was considered to consist of both the mud (clay, silty clay, clayey silt) and some of the underlying sands, and these were combined into a unit called Bay Sediments. The CPT probing and boring conducted for this investigation revealed that most of the sands underlying the upper soft mud are generally soft to moderately dense sands, silts, and clayey sands (SC) and these appear to also be Holocene-age bay deposits. Adopting the terminology from previous reports, these are shown on Figures 4-4 and 4-5 as Bay Sediments. These Bay Sediments range from about zero to 50 feet thick and also appear to represent an ancient channel. Both the Young Bay Mud and the Bay Sediments appear to pinch out to the south, where they may have been removed by dredging in the offshore area.

4.3.2.3 Merritt Sand

The Merritt Sand unit consists primarily of fine-grained sand to silty sand (SM). The shear-wave seismic velocity of the unit measured at the SFOBB was generally in the 400 to 1,650 ft/sec range indicating a dense to very dense soil layer based on the NEHRP guidelines. These sands formed as sand dunes when the sea level in the bay was lower than at present (Atwater et al., 1977). The unit can be differentiated by its color, which is brownish, and by its moderately dense to very dense nature. Marine shell and shell fragments are observed in parts of the Merritt Sand, indicating some marine reworking during the most recent sea level rise. The unit has been entirely removed by erosion in the northern part of IR Site 2. The Merritt Sand is up to 25 feet in thickness in the south and pinches out toward the northern part of the site.

4.3.2.4 San Antonio Formation

The San Antonio Formation is comprised of alluvium deposited in environments ranging from alluvial fans and flood plains to lakes and beaches. The unit is moderately dense to very dense sand and stiff to hard silt and clay. The shear-wave seismic velocity of the formation measured at the SFOBB was generally in the 400 to 1,650 ft/sec range. Similar to Merritt Sand, this is a dense to very dense soil layer. Broad channels were eroded within the surface of the upper San Antonio Formation. A sandy clay, underlain by a sandy channel fill, within the bottom of some of these channels is considered to be the Posey Sand or Posey Formation. The Posey Formation cannot be differentiated from the Merritt Sand at Alameda Point. The San Antonio Formation was encountered only in the deeper borings drilled for this investigation (see Figures 4-4 and 4-5). At Alameda Point, the upper part of the San Antonio Formation consists of medium-grained sand containing varying amounts of silt and clay, suggesting deposition in a deltaic environment. The thickness of the upper unit of the San Antonio Formation ranges from 5 to 15 feet.

4.3.2.5 Yerba Buena Mud

The Yerba Buena Mud was deposited during an interglacial period and traditionally has been referred to as the "Old Bay Mud," a homogeneous, widespread stratigraphic marker of the erosional surface of the underlying Alameda Formation, (developed during previous glacial periods). The unit is comprised primarily of a gray marine clay. However, a thin (10 to 15 feet thick) sandy, shell-rich zone is commonly found in the middle of the unit. The Yerba Buena Mud was deposited in saline bay water when sea levels were about 20 feet higher than present conditions (Sloan, 1990). The Yerba Buena Mud has been found to extend up to 2 miles inland, underlying downtown Oakland and pinching out near the Hayward Bay Area Rapid Transit station. The unit ranges in thickness from zero in the Hayward area to 125 feet near Yerba Buena Island (Atwater et al., 1977; Rogers and Figuers, 1991). The formation was encountered in borings in the area between IR Sites 1 and 2 where it is up to about 95 feet thick (see Figure 4-5).

The Yerba Buena Mud in the vicinity of Alameda Point consists of a dark greenish-gray clay. The clay is generally very plastic and commonly very stiff to hard. However, there is a wide range of blow counts indicating local softer zones. The shear-wave seismic velocity of the unit measured at the SFOBB was generally in the 650 to 800 ft/sec range, representing a stiff or dense soil layer.

4.3.2.6 Alameda Formation

The Alameda Formation was the initial unit deposited upon dissected Franciscan bedrock when the area began down-dropping between 1 million and 500,000 years ago. The unit was encountered in the deeper borings in the area between IR Sites 1 and 2 (see Figure 4-5) where it consists of silty sand and sandy silt.

Elsewhere in the vicinity, the formation includes both marine and non-marine deposits spanning several older interglacial and intervening glacial periods when the sea level was 275 to 350 feet lower than present and is the most extensive of all the late Pleistocene-age deposits (Rogers and Figuers, 1991). The formation includes shallow marine and brackish water (estuary) deposits and non-marine sediments deposited in alluvial fans, lakes, flood plains, streams, and swamps. The formation ranges from dense sand with lenses of gravel to lean hard clay. The shear-wave seismic velocity of the unit measured at the SFOBB was generally in the 800 to 1,600 ft/sec range, indicating a dense to very dense soil layer. Individual units within the Alameda Formation are typically thin and discontinuous and difficult to correlate from one borehole to another. The deposit reaches thicknesses in excess of 1,000 feet. Regional projections suggest that the formation may be about 200 to 300 feet thick below the site area.

The end of the Alameda Formation deposition, approximately 200,000 years ago appears to have been marked by a major period of erosion. During this erosional period, a series of east-west trending valleys developed along the east bay plain. By the end of the Alameda Formation deposition, the overall shape of San Francisco Bay, as it appears today, had essentially formed.

4.3.2.7 Franciscan Formation

Although the Franciscan Formation is typically a highly disturbed melange of a large number of rock types, the formation below the site appears to be quite coherent and consistent. Based on the comprehensive boring and geophysical investigations conducted for the SFOBB about 1.5 miles north of the site, the Franciscan Formation below the site comprises a sequence of interbedded graywacke sandstone, siltstone, and claystone, probably of the Alcatraz terrane. Generally, sandstone is the dominant rock type with the siltstone/claystone component of the sequence totaling about 30 percent of the formation. These rocks are very hard with shear-wave velocities generally about 10,000 ft/sec, except near the upper surface where it may be highly weathered. These weathered materials have shear-wave velocities in the 3,500 to 6,500 ft/sec range. These shear-wave velocities correspond to properties of rocks to very hard rocks.

4.3.2.8 Other Stratigraphic Units

Other stratigraphic units may occur locally throughout the east bay area, but none of these appear to occur at the site. For example, a distinctive multicolored alluvial unit, known as the Temescal Formation, overlies the San Antonio alluvium as inset terraces in east bay alluvial channels (see Figure 4-1). The Temescal Formation is almost wholly composed of silt and clay, which contains noticeable amounts of the swelling clay mineral, montmorillonite. None of these formations were encountered in site boreholes.

4.3.3 Geologic Structure

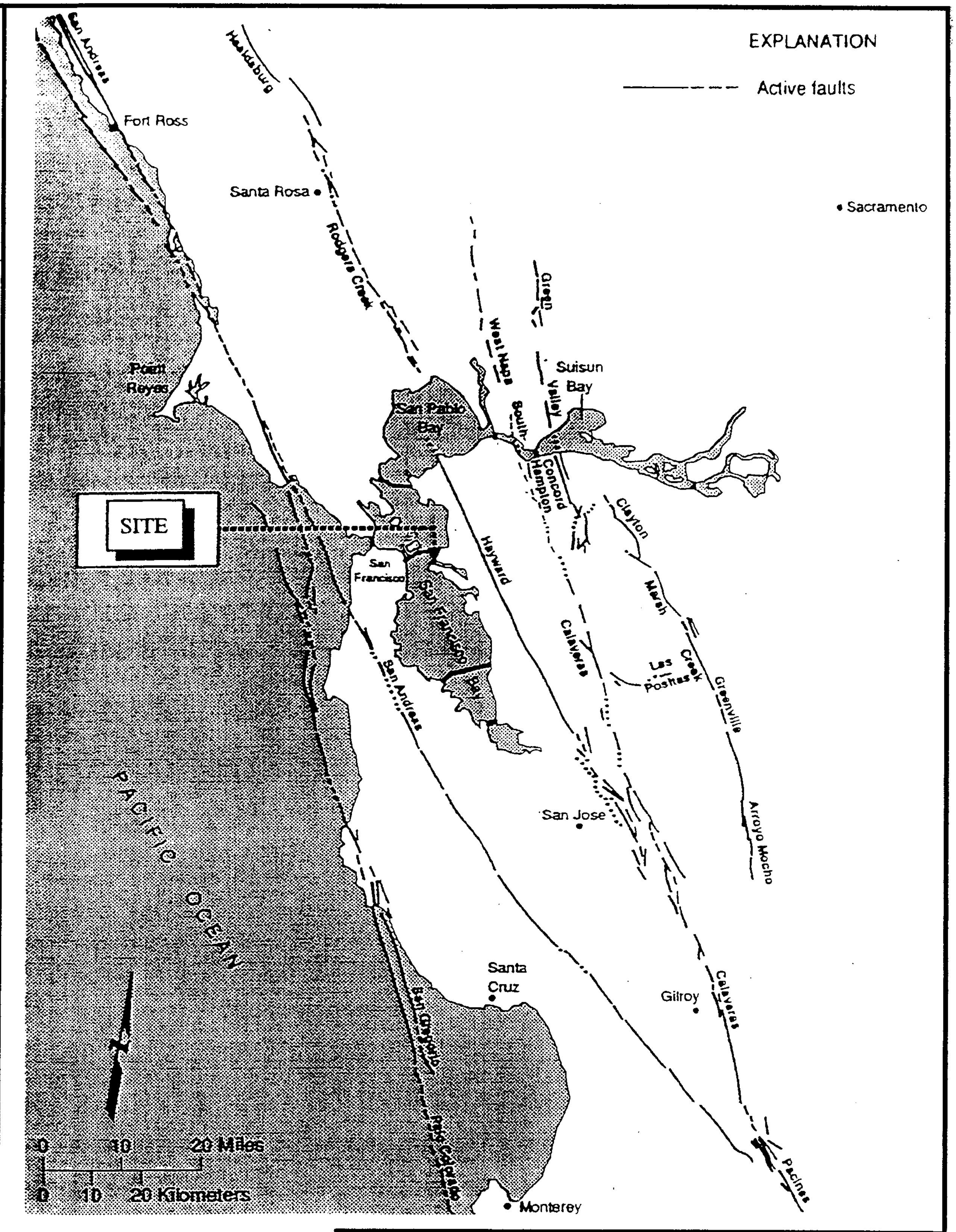
San Francisco Bay is located between two major historically active fault systems (Figure 4-6). West of the site, the San Andreas Fault juxtaposes the Jurassic/Cretaceous-age Salinian Block plutonic rocks against the Jurassic/Cretaceous-age Franciscan Complex. The San Andreas Fault is the principal bounding fault between the Pacific tectonic plate, situated to the west, and the North American tectonic plate, situated to the east.

The Bay Area depression and its bounding mountains all are of relatively recent origin. The large-scale crustal deformation that formed the depression began within only the past 3 to 4 million years, and it was not until about 1 million to 500,000 years ago that the present form of the bay could have been recognized (Page, 1992; Goldman, 1969).

The Franciscan Formation is juxtaposed against the Great Valley Sequence along the Hayward Fault, located along the eastern margin of the bay. The Hayward Fault extends northwesterly from the Santa Clara Valley, along the base of the East Bay Hills through the cities of Fremont, Oakland, Berkeley, and Richmond, about 6 miles east of the site area.

The geologic structure of the site is quite simple. The site is underlain by nearly horizontally bedded Quaternary sediments overlying the Franciscan Formation bedrock. The surface of the Franciscan Formation bedrock is irregular (see Figure 4-2), but this is due primarily to erosion. A deep bedrock trough southeast of the site area was postulated by Rogers and Figuers (1991) to be of tectonic origin. Below the site, the bedrock surface is relatively flat, but slopes slightly easterly. Regionally, the bedrock surface appears to descend southeasterly and is deepest under the southern part of the bay.

No borehole information on the bedrock below the site was available, but the comprehensive investigations for the SFOBB indicate that the bedding in the Franciscan Formation at the eastern margin of the bay consistently dips easterly at moderate angles (approximately 45 degrees). Most of this dip appears to be a result of Tertiary tectonics (Fugro-EMI, 2001a). The overlying Quaternary sediments are essentially flat lying. Most irregularities in the distribution and thickness of these materials can generally be attributed to erosion.



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SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-6
REGIONAL FAULT MAP

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4.4 GEOTECHNICAL DATA INTERPRETATION

The laboratory results summarized in Appendix H were analyzed by HAI to obtain material parameters used for geotechnical and seismic evaluations. In general, data interpretation is performed by screening out variability in results (statistical methods), correlation of engineering properties, comparison with previously published data, and engineering judgment.

4.4.1 Subsurface Soil Conditions

Subsurface soil profiles, based on information from exploratory borings and CPT soundings, were developed along and perpendicular to nearly 3,500 feet of shoreline. Figure 4-7 shows the locations of cross sections developed for IR Site 2 and the area between IR Sites 1 and 2. These profiles are shown in Figures 4-8a through 4-8i. The profiles depict interpreted stratigraphic conditions under the site. Cross sections in Figures 4-8c through 4-8h also show an assumed 4-foot-thick proposed soil cover placed on top of the existing soil cover because these cross sections are used in slope stability analyses discussed in Section 4.6.8. Slope stability analyses incorporate the proposed cover in the analysis soil profile. In addition, various profiles of penetration resistance, and classification and index property test data collected from exploratory borings are presented in Figure 4-9. Raw blow counts from a 2-inch and 2.5-inch MC drive sampler were converted to "SPT-equivalent N values" by multiplying by 0.8 and 0.6, respectively. In addition, profiles of SPT-equivalent values corrected for the effects of overburden pressure and SPT procedures are also shown in Figure 4-9.

Subsurface soil conditions at the project site can be roughly characterized in a simplified manner as Strata I through IV as discussed in the following paragraphs.

Stratum I

The top of the fill comprising most of the site occurs between an elevation +5 to +20 mean sea level (msl) and is composed of mixtures of sand, silt, and clay dredged from the surrounding bay. Existing fill is mostly classified as SP, SP-SM, with lean clay, gravel, and occasional refuse. The average moisture content and dry unit weight are 20 percent and 105 pounds per cubic foot (pcf), respectively. The average percent passing No. 200 sieve is 25. The average N value is 15 blows per foot (bpf).

Stratum II

This unit consists generally of a very dark gray clay with varying amounts of sand and silt and marine shells and organic materials. This unit is commonly referred to as Young Bay Mud. Based on the available field and soil laboratory test data, this unit can further be divided into two sub-units, namely the Offshore Bay Mud unit (Stratum IIA) and the Upland Bay Mud unit (Stratum IIB). Stratum IIA consists of predominantly very soft fat clay (CH) and silt with high plasticity. Stratum IIB in the site area predominantly consists of soft to medium stiff lean clay

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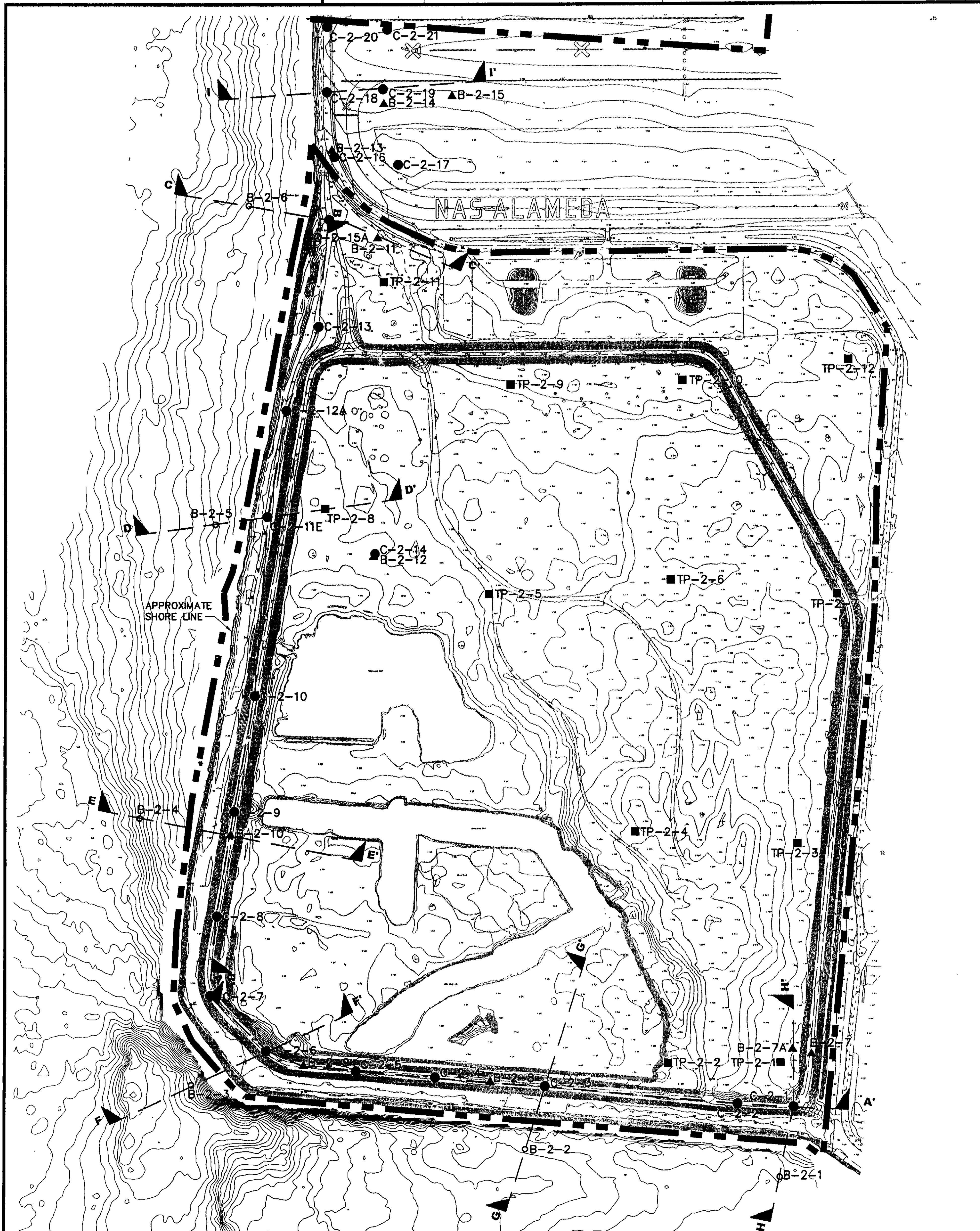

REFRENCE:
HJW-GeoSpatial, Inc., Upland topography
NAD27, NGVD29 - CCS Zone III.

LEGEND

● C	CPT SOUNDINGS
▲ B	UPLAND BORINGS
■ TP	TEST PITS
o B	OFFSHORE BORINGS
— — —	SITE BOUNDARY
F — — — F'	CROSS SECTION

N

300 150 0 300
SCALE IN FEET

Figure 4-7
CROSS SECTION LOCATION MAP

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Naval Facilities Engineering Command

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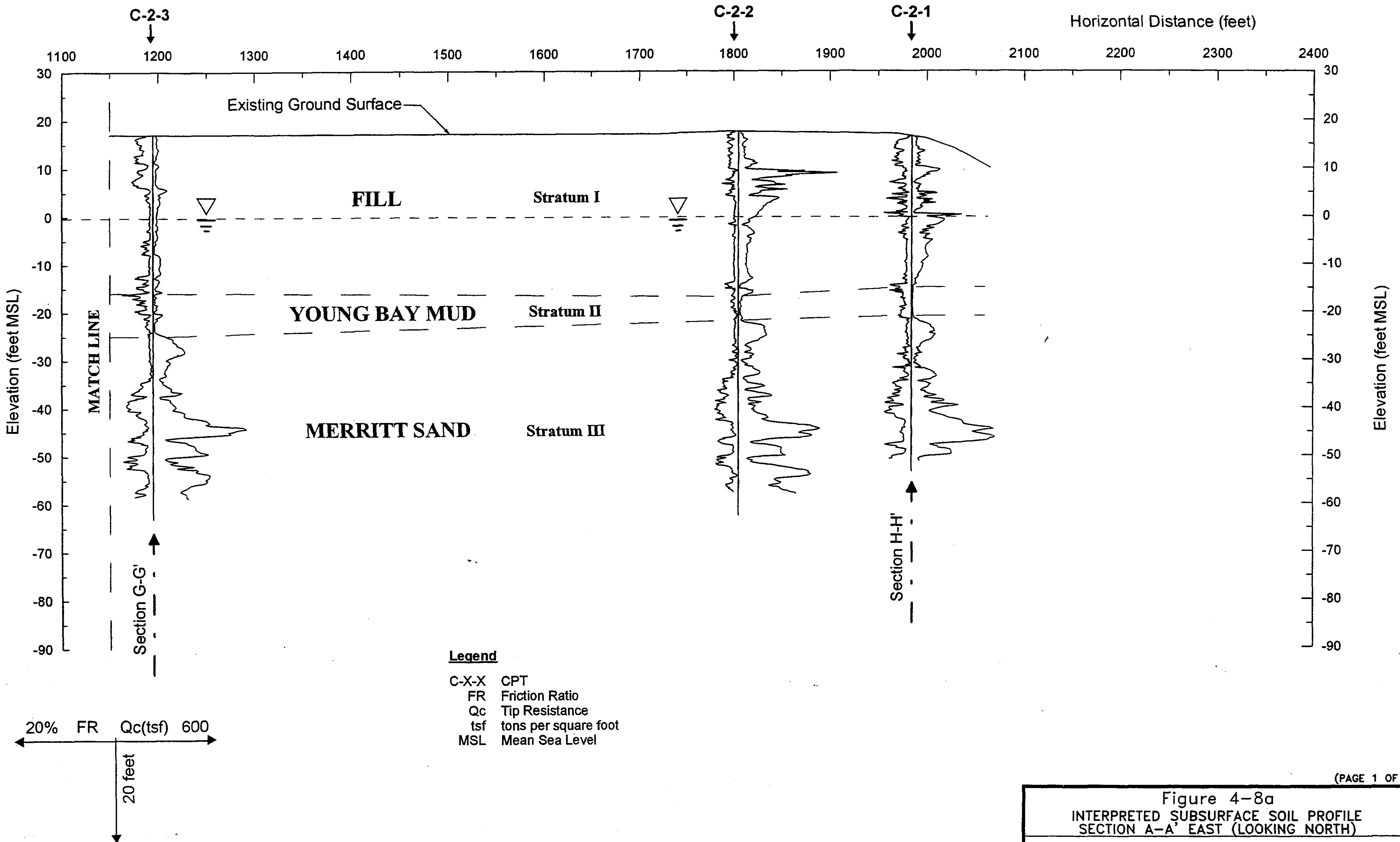

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SOURCE: HUSHMAND ASSOCIATES, INC.

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Figure 4-8a
INTERPRETED SUBSURFACE SOIL PROFILE
SECTION A-A' EAST (LOOKING NORTH)

Southwest Division
Naval Facilities Engineering Command

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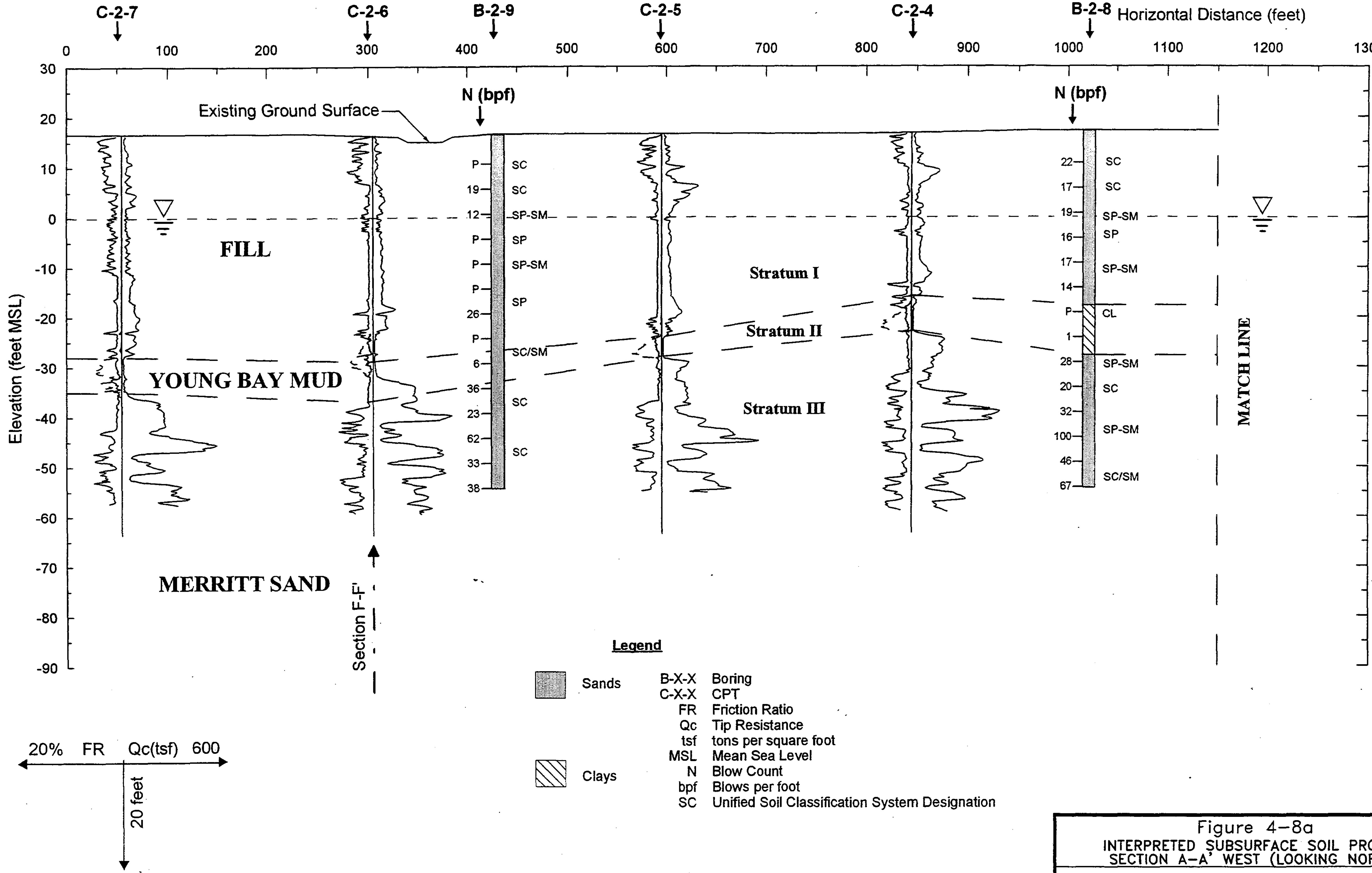

Legend

Sands	B-X-X	Boring
	C-X-X	CPT
	FR	Friction Ratio
	Qc	Tip Resistance
	tsf	tons per square foot
	MSL	Mean Sea Level
Clays	N	Blow Count
	bpf	Blows per foot
	SC	Unified Soil Classification System Designation

SOURCE: HUSHMAND ASSOCIATES, INC.

(PAGE 2 OF 2)

Figure 4–8a
INTERPRETED SUBSURFACE SOIL PROFILE
SECTION A–A' WEST (LOOKING NORTH)

Southwest Division
Naval Facilities Engineering Command

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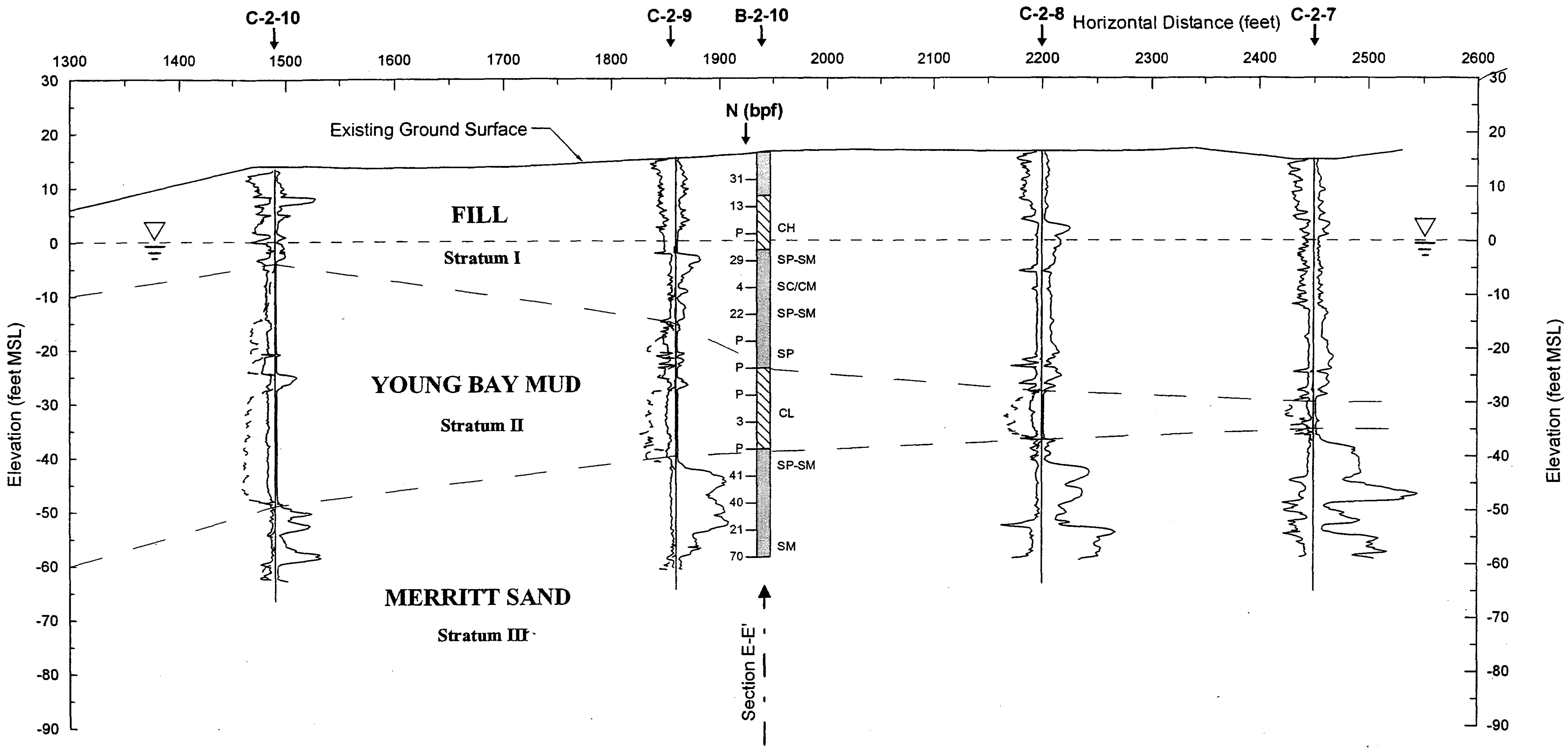

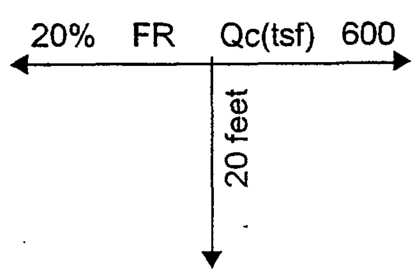

Legend

Sands

Clays

B-X-X	Boring
C-X-X	CPT
FR	Friction Ratio
Qc	Tip Resistance
tsf	tons per square foot
MSL	Mean Sea Level
N	Blow Count
bpf	Blows per foot
SP	Unified Soil Classification System Designation

SOURCE: HUSHMAND ASSOCIATES, INC.

(PAGE 1 OF 2)

Figure 4-8b
INTERPRETED SUBSURFACE SOIL PROFILE
SECTION B-B' SOUTH (LOOKING EAST)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

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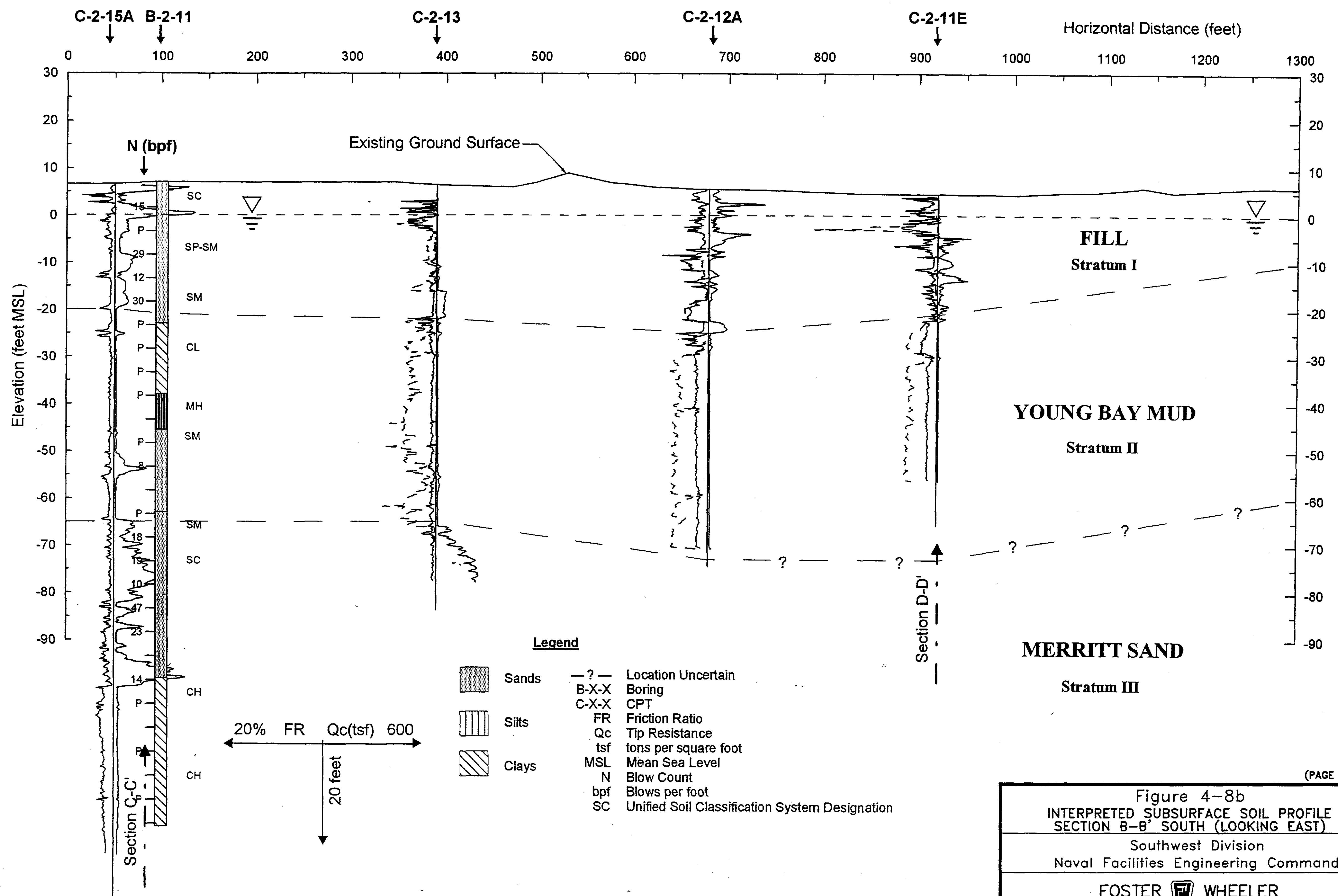

SOURCE: HUSHMAND ASSOCIATES, INC.

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Figure 4–8b
INTERPRETED SUBSURFACE SOIL PROFILE
SECTION B–B' SOUTH (LOOKING EAST)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
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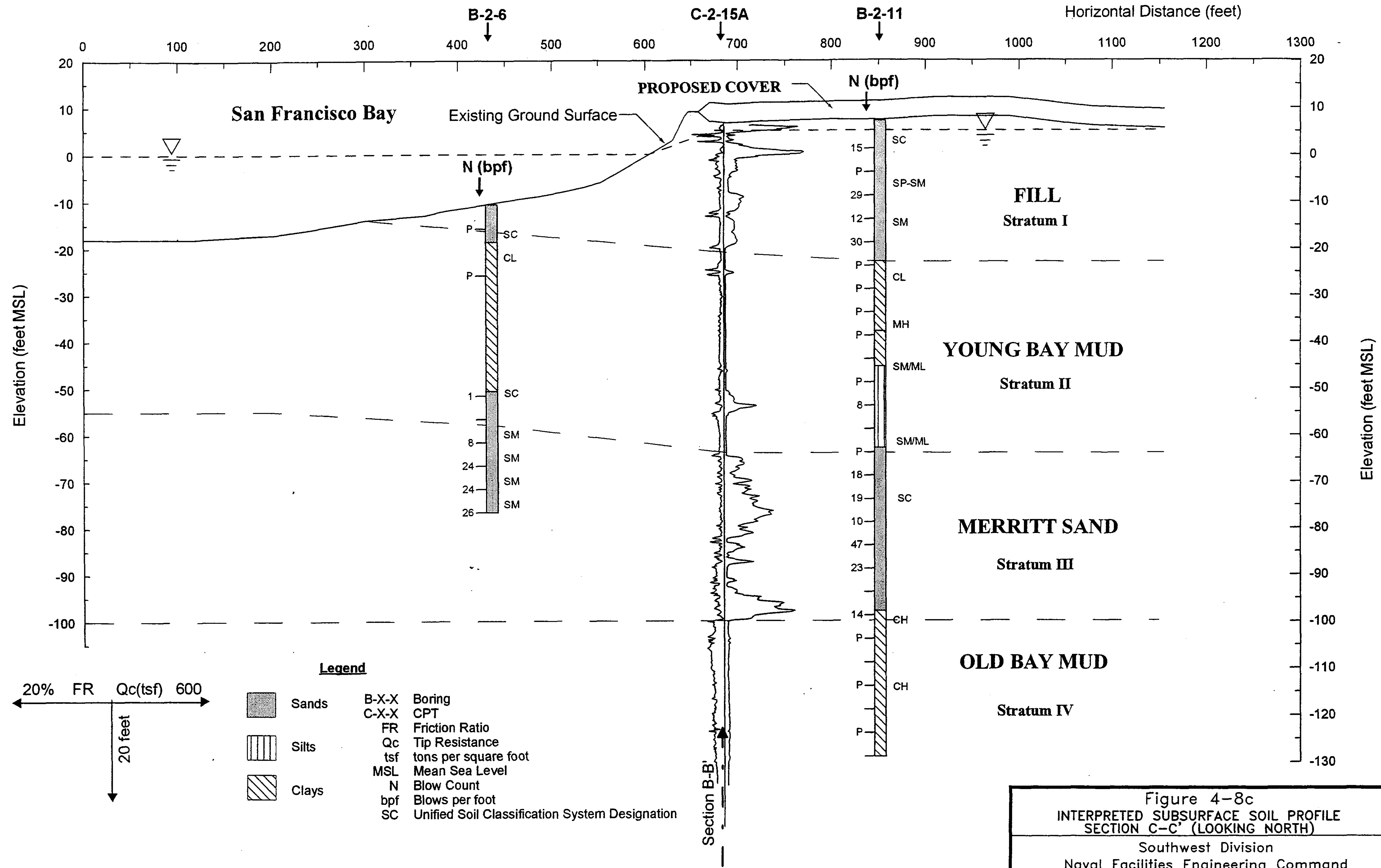

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-8c
INTERPRETED SUBSURFACE SOIL PROFILE
SECTION C-C' (LOOKING NORTH)

Southwest Division
Naval Facilities Engineering Command

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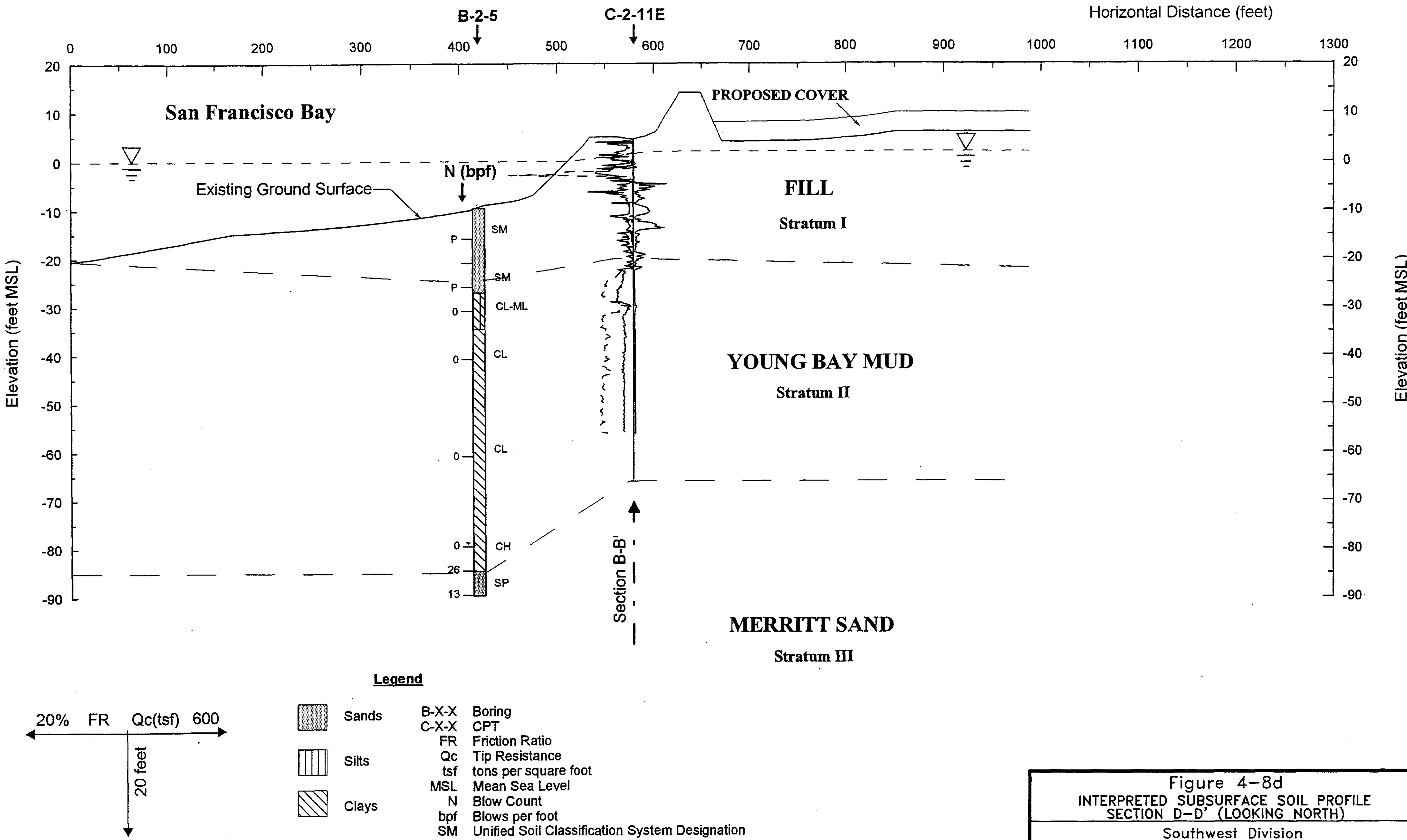

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4–8d
INTERPRETED SUBSURFACE SOIL PROFILE
SECTION D–D' (LOOKING NORTH)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

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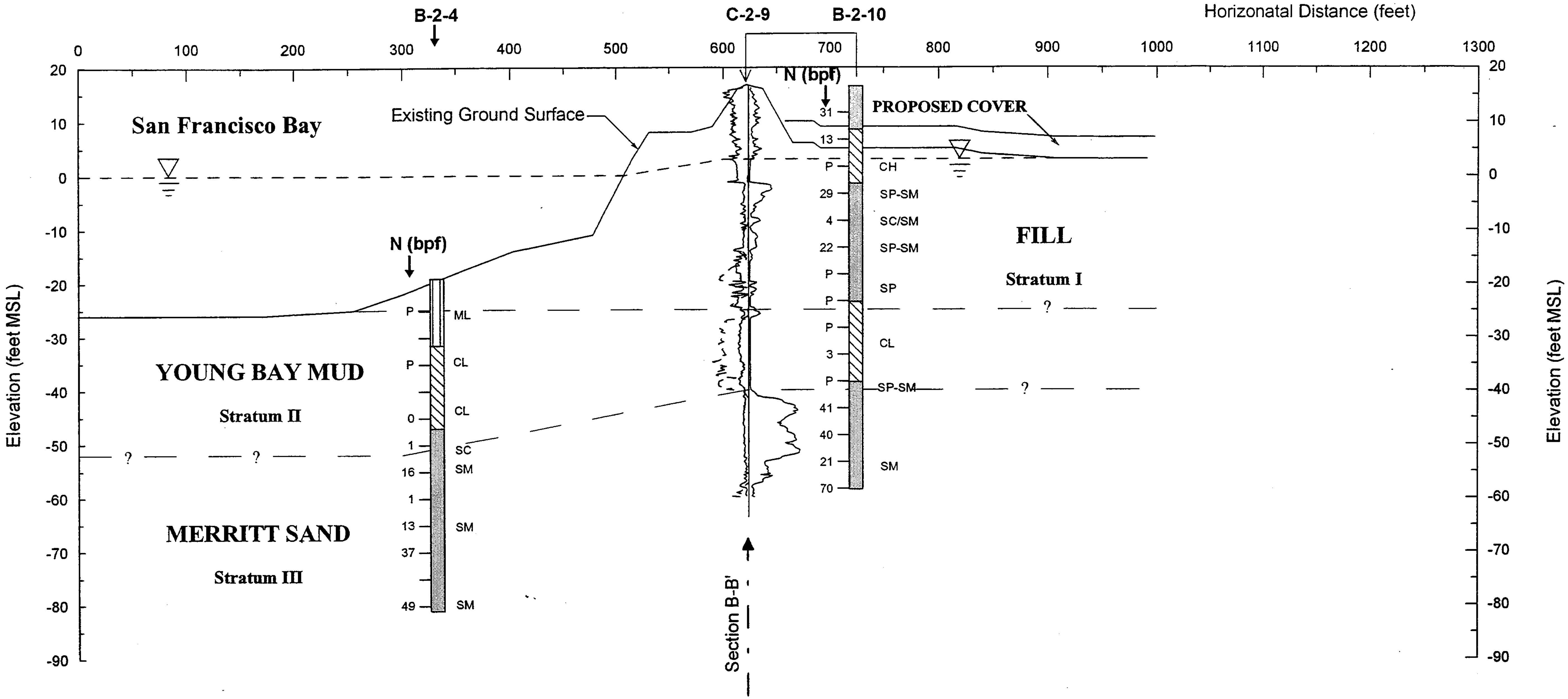

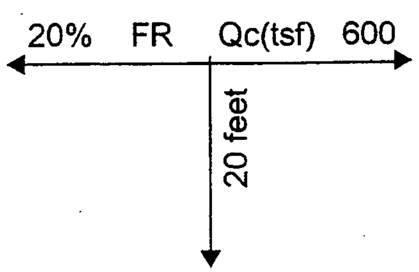

Legend

Sands	— ? — Location Uncertain
	B-X-X Boring
	C-X-X CPT
Silts	FR Friction Ratio
	Qc Tip Resistance
	tsf tons per square foot
Clays	MSL Mean Sea Level
	N Blow Count
	bpf Blows per foot
	SM Unified Soil Classification System Designation

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-8e
INTERPRETED SUBSURFACE SOIL PROFILE
SECTION E-E' (LOOKING NORTH)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWING NO: 03289948e.DWG
DCN: FWSD-RAC-03-2899
CTO: #0054
APPROVED BY: AL
CHECKED BY: TL
REV: REVISION 0
DRAWN BY: MD
DATE: 10/29/03

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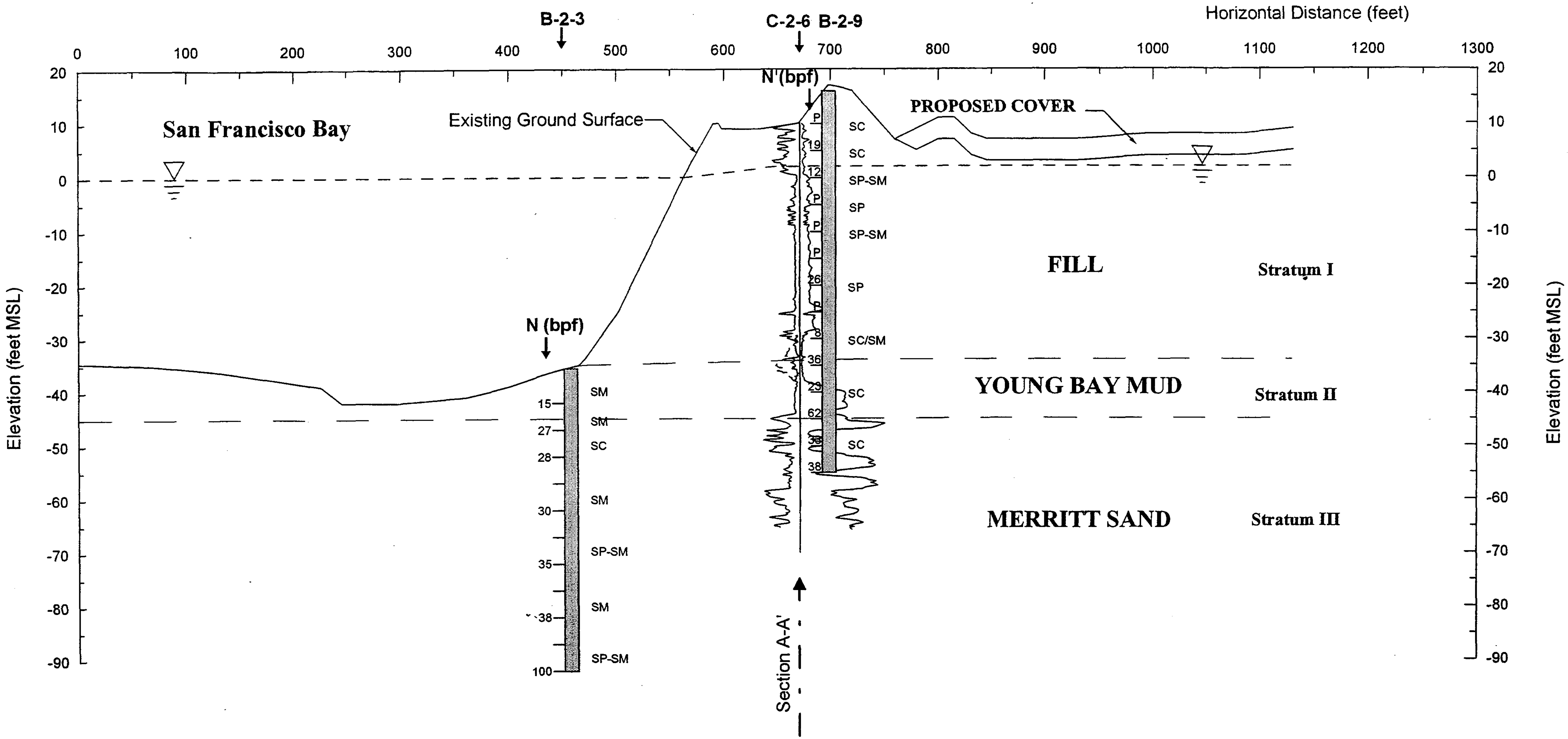

Legend

Sands

B-X-X	Boring
C-X-X	CPT
FR	Friction Ratio
Qc	Tip Resistance
tsf	tons per square foot
MSL	Mean Sea Level
N	Blow Count
bpf	Blows per foot
SC	Unified Soil Classification System Designation

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-8f
INTERPRETED SUBSURFACE SOIL PROFILE
SECTION F-F' (LOOKING NORTH)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWING NO: 03289948f.DWG
DCN: FWSD-RAC-03-2899 CTO: #0054
APPROVED BY: AL
CHECKED BY: TL REV: REVISION 0
DRAWN BY: MD DATE: 10/29/03

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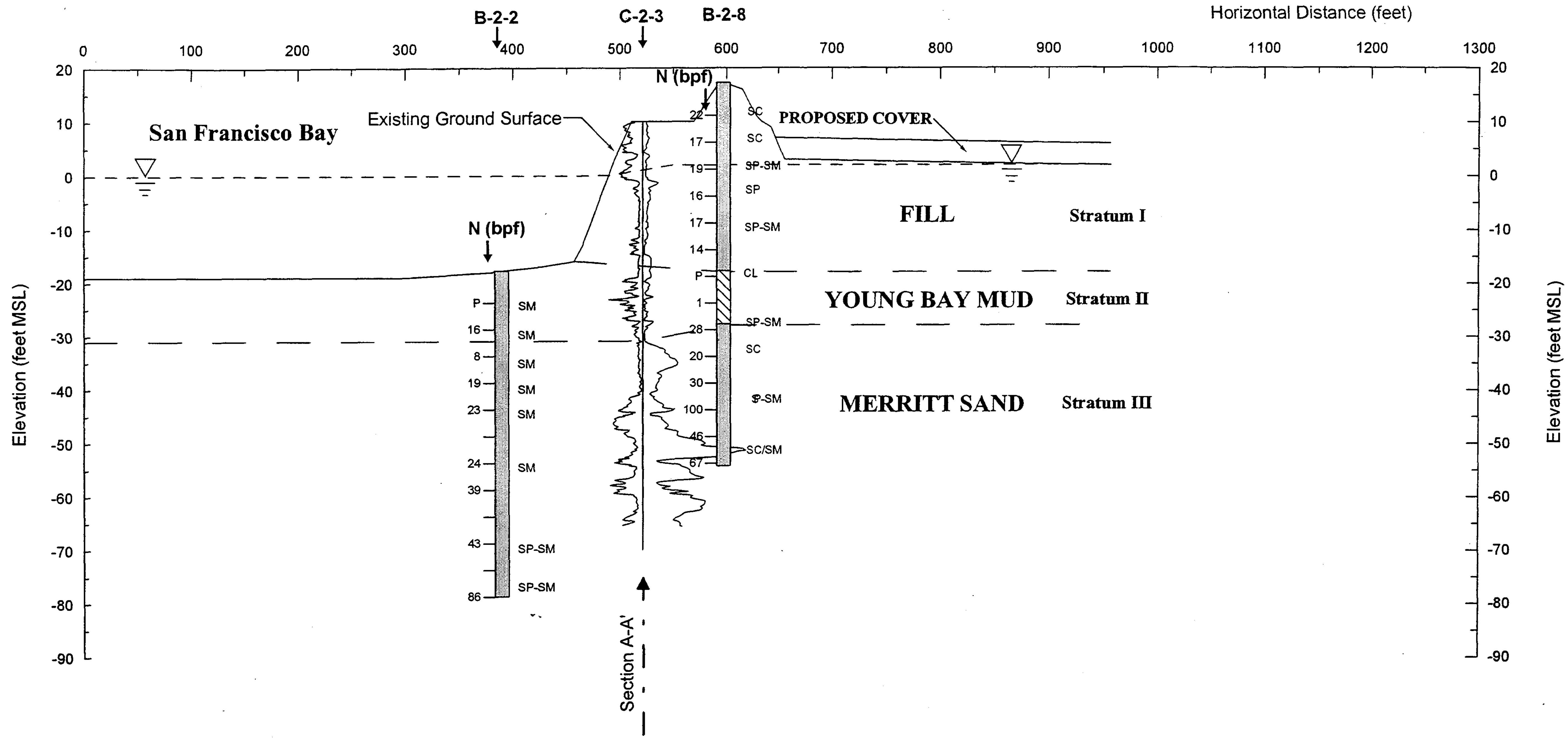

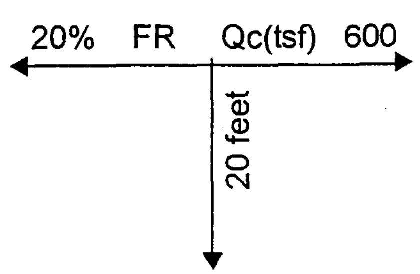

Legend

Sands

Clays

B-X-X	Boring
C-X-X	CPT
FR	Friction Ratio
Qc	Tip Resistance
tsf	tons per square foot
MSL	Mean Sea Level
N	Blow Count
bpf	Blows per foot
SC	Unified Soil Classification System Designation

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4–8g
INTERPRETED SUBSURFACE SOIL PROFILE
SECTION G–G' (LOOKING NORTH)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

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DCN: FWSD-RAC-03-2899
CTO: #0054
APPROVED BY: AL
CHECKED BY: TL
REV: REVISION 0
DRAWN BY: MD
DATE: 10/29/03

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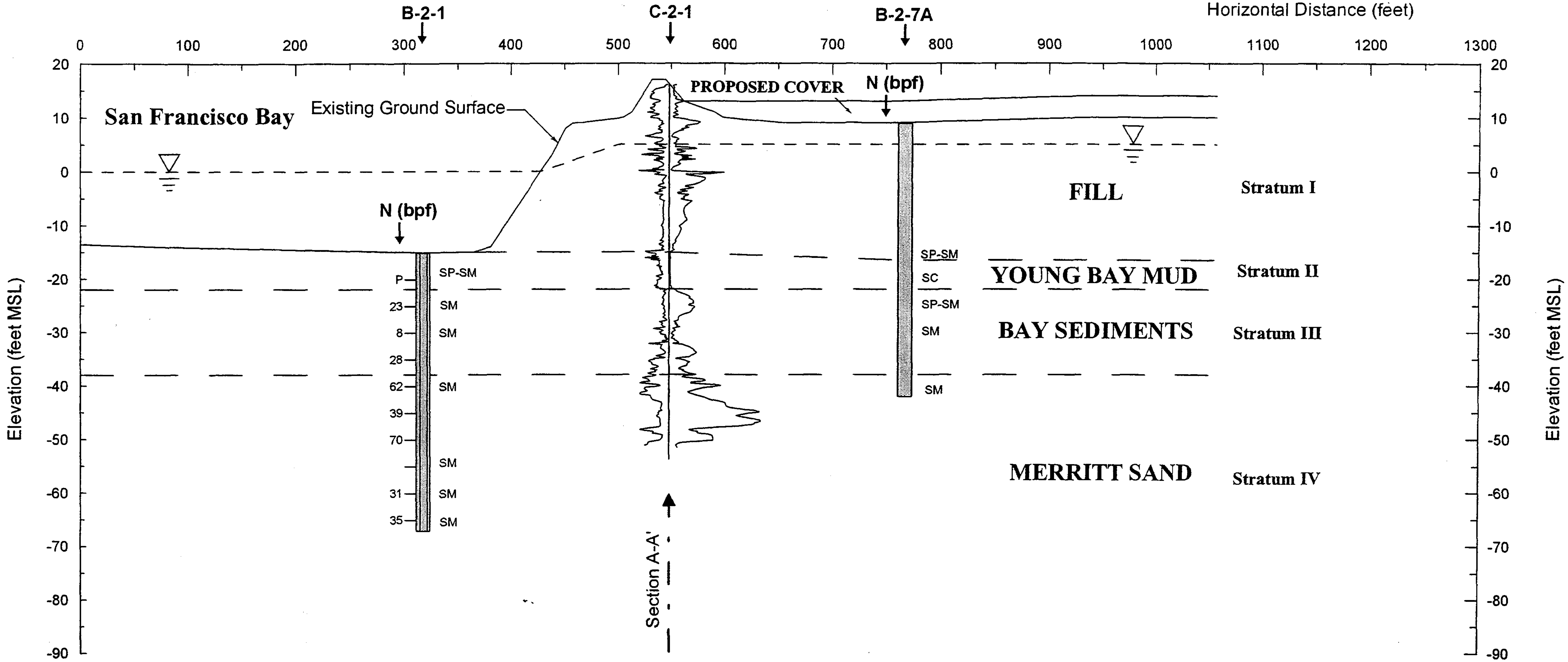

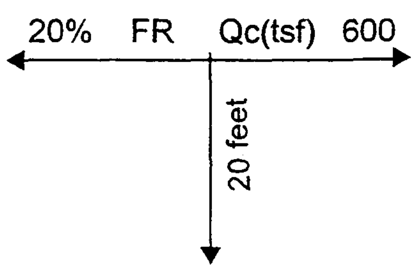

Legend

Sands

Silts

B-X-X	Boring
C-X-X	CPT
FR	Friction Ratio
Qc	Tip Resistance
tsf	tons per square foot
MSL	Mean Sea Level
N	Blow Count
bpf	Blows per foot
SM	Unified Soil Classification System Designation

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4–8h
INTERPRETED SUBSURFACE SOIL PROFILE
SECTION H–H' (LOOKING NORTH)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

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DCN: FWSD-RAC-03-2899
CTO: #0054
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REV: REVISION 0
DRAWN BY: MD
DATE: 10/29/03

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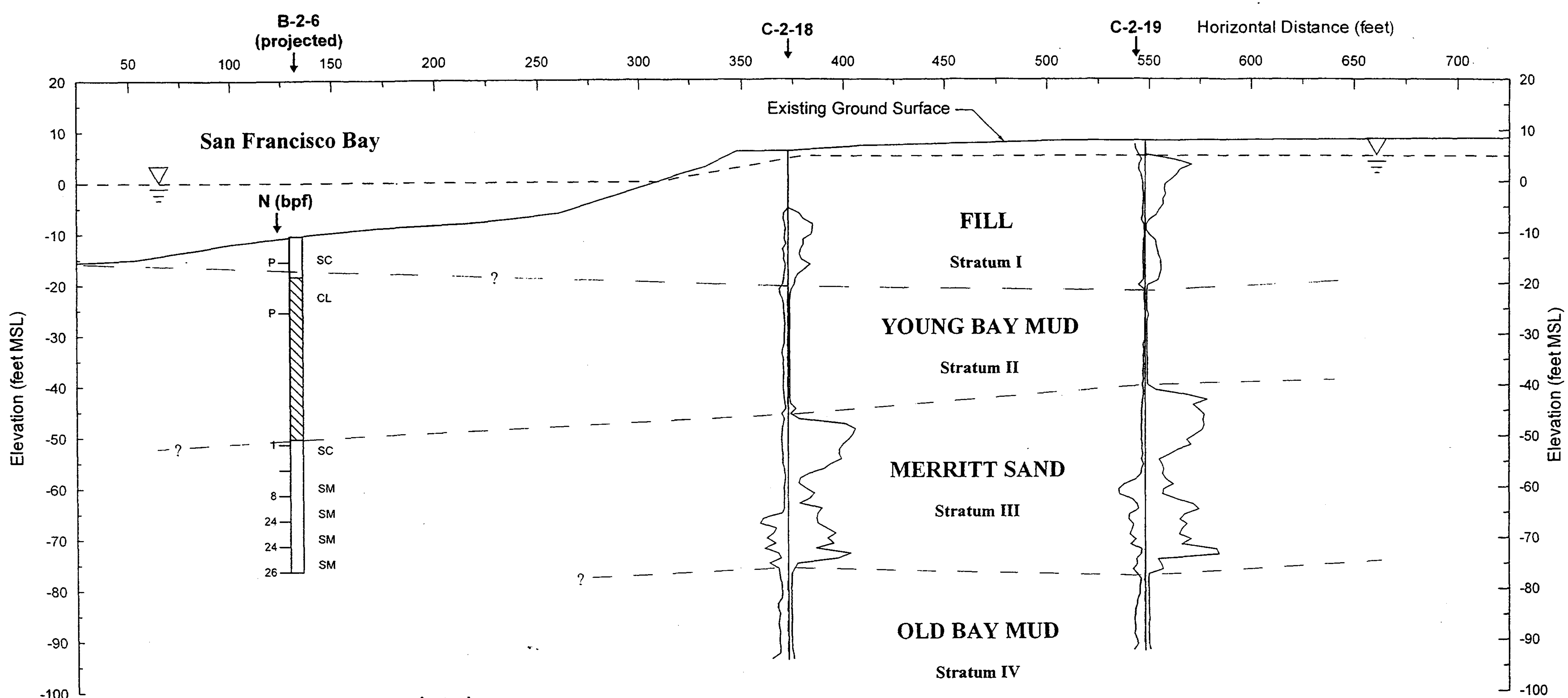
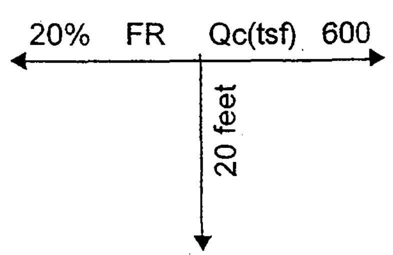


Legend

Sands		— ? —	Location Uncertain
		B-X-X	Boring
		C-X-X	CPT
		FR	Friction Ratio
		Qc	Tip Resistance
		tsf	tons per square foot
Clays		MSL	Mean Sea Level
		N	Blow Count
		bpf	Blows per foot
		SC	Unified Soil Classification System Designation



**NOTE: UPPER 11 FEET OF CONE PENETRATION TESTS WERE PRE-PUNCHED IN SOME CASES.*

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4–8i
INTERPRETED SUBSURFACE SOIL PROFILE
SECTION I–I' (LOOKING NORTH)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

DRAWING NO: 03289948i.DWG
DCN: FWSD-RAC-03-2899
CTO: #0054
APPROVED BY: AL
CHECKED BY: TL
REV: REVISION 0
DRAWN BY: MD
DATE: 10/29/03

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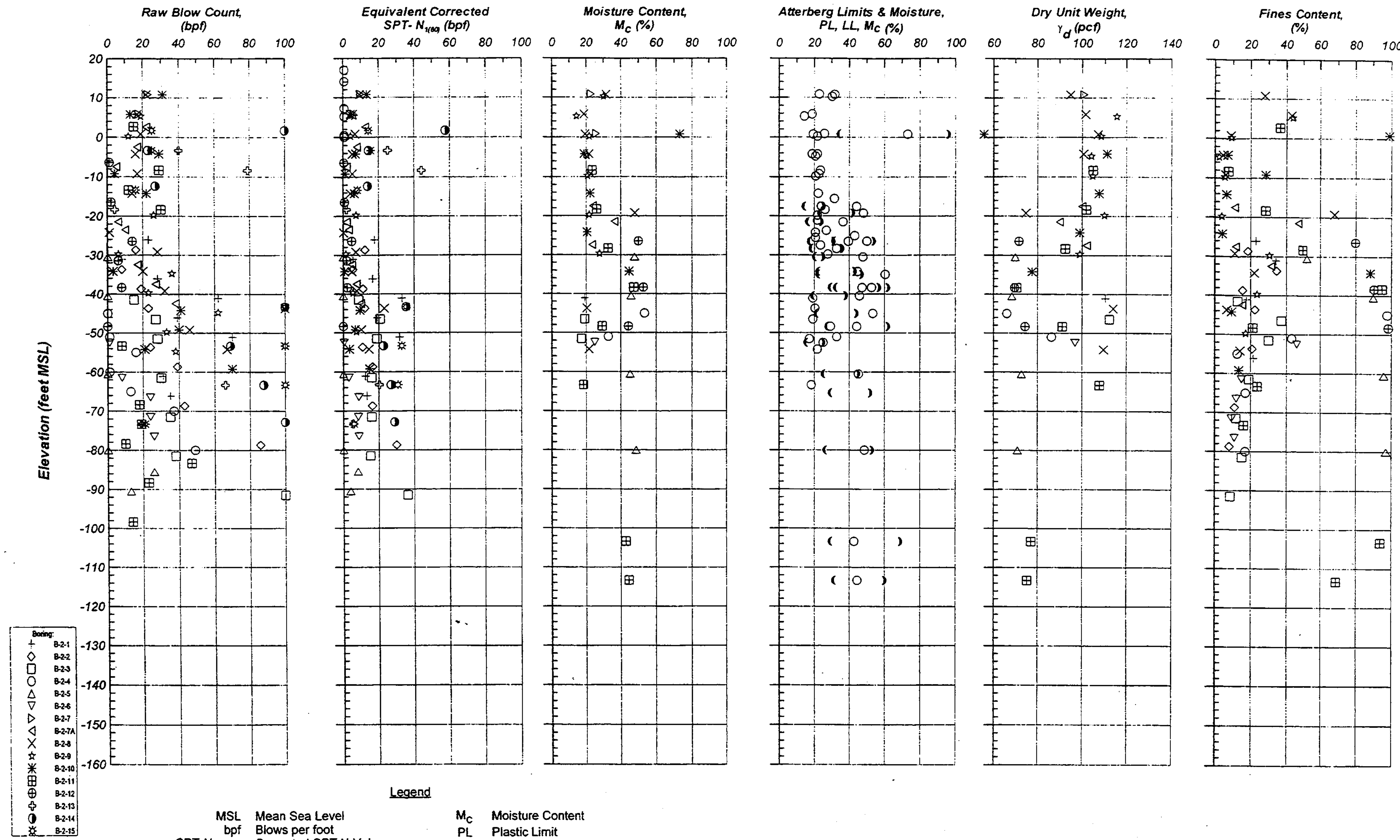

Legend

MSL	Mean Sea Level	M_c	Moisture Content
bpf	Blows per foot	PL	Plastic Limit
SPT-$N_{1(60)}$	Corrected SPT N-Value	LL	Liquid Limit
SPT	Standard Penetration Test	γ_d	Dry Unit Weight
N-Value	Blow Count	pcf	pounds per cubic foot

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-9
SOIL PROPERTY CHARACTERIZATION DATA VERSUS ELEVATION (feet MSL)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER ENVIRONMENTAL CORPORATION

DRAWING NO: 03289949.DWG
DCN: FWSD-RAC-03-2899 CTO: #0054
APPROVED BY: AL
CHECKED BY: TL REV: REVISION 0
DRAWN BY: MD DATE: 10/29/03

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and silty clay/clayey silt. The thickness of Stratum IIA (offshore) ranges from 10 to 60 feet while the thickness of Stratum IIB (upland) in the site area is about 5 to 50 feet. Both sub-units are mostly normally consolidated to slightly underconsolidated with in situ moisture content values relatively close to their liquid limits. The offshore sub-unit contains soils with a slightly higher degree of underconsolidation. However, a distinct difference in the degree of consolidation between offshore and upland soils is not apparent. Based on available laboratory test data, the approximate average soil index properties of this unit are as follows:

Property	Upland and Offshore
Average Moisture Content (%)	50
Average Dry Unit Weight (pcf)	75
Liquid Limit (%)	35 to 65
Plastic Limit (%)	10 to 30
Liquidity Index (%)	1.2 to 1.5

As shown in the CPT data, Stratum IIB (upland) is classified as sensitive fine-grained soils and is subject to strength degradation after cyclic loading (for example, earthquake loading). Based on field and laboratory test results (Appendix G and H), Stratum IIA (offshore) is as sensitive as Stratum IIB (upland).

__Stratum III__

This unit comprises the Merritt Sand/Bay Sediment, mostly classified as dense fine-grained sand (SC, SM, SP, SP-SM), having an average moisture content and dry unit weight of 15 and 120 pcf, respectively. The top of this layer occurs between elevations -20 to -85 bgs. The average passing No. 200 sieve value is 16 percent.

__Stratum IV__

The Old Bay Mud in the vicinity of Alameda Point consists of stiff to hard, dark greenish-gray, very plastic silty clay (CL, CH). The top of this clay layer occurs at a minimum elevation of -75 to -100 feet msl.

4.4.2 Groundwater

Groundwater was encountered in the CPT soundings at elevations ranging from approximately +4 to +5 feet msl. Groundwater levels are subject to seasonal and tidal variations and should be expected to change on the order of several inches to a few feet. For static (long-term) slope stability purposes, it was assumed, conservatively, groundwater levels ranging from +2 to

+5 feet msl elevation upland, and at zero (msl) elevation offshore. For dynamic slope stability analysis, groundwater levels were conservatively assumed to be +2 msl elevation upland and at zero (msl) elevation offshore. Assumed groundwater levels for dynamic conditions are a few feet lower than those for static (long-term) condition since the probability of having simultaneously an earthquake and high water levels is very low.

4.4.3 Material Design Parameters

Based on field observations, test results, and preliminary analyses, four distinct geologic units exist at IR Site 2. These include fill materials, Young Bay Mud (offshore and upland), dense sands, and stiff clays. A summary of the geotechnical design parameters for each geologic unit was prepared by HAI and is presented in Table 4-6a. Information provided in the table includes: 1) available field data, 2) classification and index properties, and 3) engineering properties. A discussion of shear strength properties presented in Table 4-6a is provided in Section 4.6.8.

4.5 GEOTECHNICAL ENGINEERING ANALYSES

Geotechnical engineering analyses were performed to supplement field and laboratory testing. Issues addressed in the following sections include bearing capacity failure potential, hydraulic performance of existing soil cover, settlements, and static slope stability. The concern regarding bearing capacity failure was specifically cited in the statement of work issued by the Navy, dated August 14, 2001. The other issues mentioned above were described in the Work Plan (FWENC, 2002b) as being potentially important for geotechnical evaluations.

4.5.1 Bearing Capacity

IR Site 2 consists of the landfill, wetland, coastal margin, and the interior margin (see Figure 2-1). Since the site is currently used as a bird and wildlife sanctuary and is proposed for transfer to the United States Fish and Wildlife Service (USFWS) for use as a National Wildlife Refuge, no significant additional loads are expected in the wetland and coastal margin areas. However, a landfill cap and additional fill material could be placed at the landfill and interior margin areas. The future landfill cap and/or additional fill will be spread out over the entire site with gradual changes in thickness, yielding relatively uniform loads. Consequently, foundation soils would not be subjected to relatively high concentrated loads, which may lead to bearing capacity failure. The mechanism for a bearing failure is soil heaving. Without this mechanism, bearing capacity failures from placement of a landfill cap or additional fill material for grading are not considered a significant hazard at this time. Localized bearing capacity failures may become a concern during construction due to stockpiling of cap/fill materials on the site. This issue can be addressed by temporarily storing fill materials away from the shoreline slopes where concentrated (stockpile) loads can create a bearing failure hazard.

TABLE 4-6a

SUMMARY OF MATERIAL DESIGN PARAMETERS

Generalized Stratum	Units	I	IIA	IIB	III	IV
Description		**Fill Materials**	**Offshore** Soft Harbor Sediments, Young Bay Mud	**Upland** Soft Harbor Sediments, Young Bay Mud	**Dense Sands (Merrit Sand)**	**Stiff Clays (Old Bay Mud)**
Unified Soil Classification		Very loose to medium dense sands (SM, SC, SP, SP-SM), with occasional layers/lenses of fine-grained soils, pieces of gravel and refuse	NC to slightly UC fine-grained soils: ML, MH, CL, CH	NC to slightly UC fine-grained soils: ML, MH, CL, CH	Medium dense to very dense sands (SM, SC, SP, SP-SM)	Stiff to very stiff clays (CH, CL)
Borings Providing Data	No	B4 through B11	B1 through B6	B1 through B11	B1 through B11	B1, C752, C753
Typical Elevation Range[1]	feet msl	-15 to +20	-55 to -15	-55 to -10	-85 to -45	Below – 75
Typical Thickness[1]	feet	20 to 45	10 to 60	5 to 50	20 to 50	> 15
Raw SPT-N Values - Mean ± Std. Deviation	bpf	15 ± 10	4 ± 3	4 ± 3	40 ± 20	15 ± 5
Raw CPT Tip Resistance (Q_c) Values	tsf	75 ± 25	N/A	8 ± 3	200 ± 100	20 ± 10
Volumetric/Gravimetric Relationships						
Total Unit Weight	pcf	130	110	110	131	125
Moisture Content	%	20	50	50	20	45
Dry Unit Weight	pcf	105	75	75	110	86
Void Ratio		0.57	1.29	1.29	0.44	1.02
Specific Gravity		2.65	2.75	2.75	2.65	2.75
Atterberg Limits						
Liquid Limit, LL (range)	%	No plastic fines	36 to 65	36 to 65	No plastic fines	60 to 80
Plastic Limit, PL (range)	%	No plastic fines	15 to 25	15 to 25	No plastic fines	25 to 35
Plasticity Index, PI (range)	%	No plastic fines	10 to 30	10 to 30	No plastic fines	30 to 40
Liquidity Index, LI (range)	%	No plastic fines	1 to 2	1 to 2	No plastic fines	0.3 to 0.4
Gradation Characteristics						
Fines Content (< 74 microns), FC	%	25 ± 20	50 to 100	50 to 100	15 ± 5	50 to 100
CD Shear Strength Parameters - Static Stability[2]						
Peak Internal Friction Angle (CD)	degrees	32	25	25	38	30
Peak Cohesion Intercept (CD)	psf	0	0	0	0	0
Residual Internal Friction Angle (CD)	degrees	30	25	25	38	30

TABLE 4-6a

SUMMARY OF MATERIAL DESIGN PARAMETERS

Generalized Stratum	Units	I	IIA	IIB	III	IV
Description		**Fill Materials**	**Offshore** Soft Harbor Sediments, Young Bay Mud	**Upland** Soft Harbor Sediments, Young Bay Mud	**Dense Sands (Merrit Sand)**	**Stiff Clays (Old Bay Mud)**
CU Shear Strength Parameters – Seismic (Pseudo-Static) Stability[3]						
SHANSEP's Normalized Static Pre-EQ Undrained Shear Strength $(Su/\sigma'_{vo})'_{NC}$		N/A	0.2 (S_U = 300 psf)	0.2 (S_U = 500 psf)	N/A	0.3 (S_U = 1,300 psf)
SHANSEP's Normalized Post-EQ Undrained Shear Strength $(Su/\sigma'_{vo})_{NC}$		N/A	0.16	0.16	N/A	0.3
Post-Earthquake/Liquefaction Undrained Shear Strength $(S_U)_r$	psf	300	150	400	N/A	1000
Compressibility Characteristics						
Compression Index, Cc		0.08	0.13 to 0.35	0.13 to 0.35	0.025	N/A
Coefficient of Consolidation, Cv	feet²/year	N/A	4 to 10	4 to 10	N/A	N/A
Secondary Compression Index, Cα		N/A	0.01	0.01	N/A	N/A

Notes:

1 Elevations and thicknesses of generalized soil layers vary considerably across the site as shown in interpreted subsurface soil profiles along Cross Sections A-A' through I-I' in Figures 4-8a to 4-8i of the report.

2 CD shear strength properties of sands and clays were derived from results of laboratory tests (Appendix H). Direct shear tests and CU triaxial shear tests with pore pressure measurement were performed to estimate CD shear strength properties used in long-term static stability analyses.

3 CU shear strength parameters of Young Bay Mud and Old Bay Mud were estimated based on the results of field and laboratory tests performed for this project and a survey of the published data (Furgro-EMI, 2001a; 2001b; Pyke, 1989; Ramanujam et al., 1978). The Stress History and Normalized Soil Engineering Properties (SHANSEP) method (Ladd, 1974; 1991), laboratory and published data, and correlations with Liquidity Index (LI) and Plasticity Index (PI) were used to provide estimates of the ratio.

bpf – blows per foot
σ'_{vo} – initial effective vertical pressure (overburden)
Cc – clay content
CD – consolidated-drained
CH – fat clay
CL – lean clay
CPT – cone penetrometer test
CU – consolidated-undrained
EQ – earthquake
FC – fines content
MH – high plasticity
ML – sandy sitls/silty clays
msl – mean sea level
N/A – not applicable
NC – normally consolidated
pcf – pounds per cubic foot
psf – pounds per square foot
Q_c – cone penetration tip resistance
SC – clayey sand
SHANSEP – Stress History and Normalized Soil Engineering Properties
SPT-N – standard penetration test, N value
S_u – undrained shear strength, used for end-of-construction stability evaluations
$(S_u)_r$ – residual undrained shear strength, used for static post-earthquake stability evaluations
(Su/σ'_{vo}) – undrained shear strength ratio, where σ'_{vo} is the initial effective overburden pressure
SM – silty sand
SP – poorly graded sand
Std. – standard
tsf – tons per square foot
UC – under-consolidated

The presence of waste materials throughout IR Site 2 will also impact the bearing capacity of soils. Since only relatively uniform loads are expected at the site, bearing capacity failure is not considered a concern. However, the presence of waste materials will have an impact on the ground settlements from placement of a landfill cap and additional fill material. Immediate and long-term ground settlements are addressed in Section 4.5.3.

4.5.2 Hydraulic Performance of Existing Soil Cover

Test pit explorations were conducted to determine the thickness and type of the existing soil cover over the landfill and interior margin areas (see Figure 2-1). Results of test pit explorations show that there was no consistency in the existing soil cover. The thickness of the existing soil covers varied from 2 inches (TP-2-5) to 2 feet (TP-2-4, TP-2-6, and TP-2-12) over the refuse. Significant amounts of construction debris such as concrete fragments, pipe, gravel, asphalt, and brick were found just 2 to 3 inches bgs (TP-2-5, TP-2-7, TP-2-9, and TP-2-11). The predominant soil cover type was poorly graded sand (SP) and silty sand (SM). The sand size was classified as medium to fine and generally loose with some trace of gravel. Overall, the existing soil cover was found to be inconsistent and poorly compacted. Therefore, the material was determined to be unsuitable for use as part of the final cover and no saturated hydraulic conductivity tests were conducted. Instead, it is recommended to evaluate any future fill material by conducting laboratory testing and performing percolation modeling.

4.5.3 Settlements

The geological units identified at IR Site 2 include: Fill, Young Bay Mud (Bay Sediments), Merritt Sand, San Antonio Formation, Yerba Buena Mud (Old Bay Mud), Alameda Formation, and Franciscan Formation (see Geological Cross Sections in Figures 4-4 and 4-5). Ground settlements from future placement of a landfill cap or additional fill for grading purposes will occur mainly from 1) elastic settlement of the fill layer and 2) consolidation/compression of the Young Bay Mud layer. Settlements from the Merritt Sand, San Antonio Formation, Alameda Formation, and Franciscan Formation are considered negligible since they are very dense/stiff compared to the other soil layers (see CPT data in Appendix B).

Elastic settlements of the fill layer were estimated using the theory of elasticity. In this method, the fill layer was subdivided into sub-layers based on similar strength parameters (tip resistance). An averaged elastic modulus for each sub-layer was obtained by correlation with the tip resistance (Das, 1990). The strain (settlement) of each sub-layer was then calculated and the total elastic settlement was obtained by adding the settlements from each sub-layer. This calculation was made for each CPT location.

Settlements of the Young Bay Mud and Old Bay Mud layers would consist of primary consolidation, secondary compression, and elastic/immediate settlement. Consolidation settlements were estimated using one-dimensional consolidation theory (Terzaghi as described by Coduto, 1994). This method was considered appropriate since the loading condition is generally uniform and extends laterally throughout the site. Secondary compression settlement due to creep, compression, and decomposition of organic matter was calculated for a period of 30 years after load application and added to the overall settlement. Elastic/immediate settlement is only a concern for unsaturated and highly over-consolidated clays. Since the Young Bay Mud layer is under the water table and classified as normally consolidated to slightly under-consolidated, elastic/immediate settlement is not considered to for this type of material.

The final thickness of the landfill cap has not been determined at this time. Therefore, a 4-foot-thick cap was assumed for settlement evaluation. In addition, a maximum additional fill thickness of 10 feet was assumed to maintain proper drainage and slope design. The additional fill materials placed at the site will cause uneven loading and deformation. Total settlements were calculated for the maximum assumed loading (landfill cap with additional fill) and the minimum assumed loading (landfill cap only). Table 4-6b presents a summary of the settlement results for each CPT location considered. It includes estimates for elastic, consolidation, and secondary compression settlements at each CPT location. The estimated time to complete primary consolidation is also provided. The maximum total settlements calculated are 12.86 and 33.87 inches, respectively, for the two loading conditions. Calculation details are presented in Appendix K.

The calculated settlements on top of the perimeter berm (CPT locations C-2-1 to C-2-10) result mainly from elastic settlements and are significantly less than those calculated adjacent to the berm (C-2-11 to C-2-15). The settlements on top of the berm were calculated using an assumed loading (4 to 14 feet of fill); however, no significant settlements are anticipated because no additional fill is planned for placement on top of the berm.

A landfill cap and additional fill could be placed at the landfill area in the future. Larger settlements are expected in this area due to the presence of a thick Young Bay Mud layer (up to 40 feet) and sensitive fine-grained material just below the ground surface. However, these settlements are expected to occur over a long period of time (40 years or more) after load application because of the low permeability of the Young Bay Mud and fine-grained material [Coefficient of consolidation, $C_v = 0.000032$ square inches per second (in^2/sec)]. The relatively high overall and varying ground settlements warrant further evaluations once the final cover design has been determined.

In the area between IR Sites 1 and 2, the Old Bay Mud layer (see Figure 4-8i) existing beneath the Merritt Sand extends up to approximately 200 feet bgs. This layer, which consists of fine-grained sensitive soil similar to the Young Bay Mud, contributes additional consolidation settlement. The maximum total settlements calculated in this area (C-2-16 to C-2-21) are 8.47 and 27.5 inches, respectively, for the two loading conditions considered. The difference in total settlements for the two loading conditions was estimated to be 13 to 19 inches. These settlements

TABLE 4-6b

SUMMARY OF SETTLEMENT CALCULATIONS

CPT No.	Elastic (landfill cap) (inch)	Elastic (cap+fill) (inch)	Consolidation (landfill cap) (inch)	Consolidation (cap+fill) (inch)	Secondary Compression (inch)	Total (landfill cap) (inch)	Total (cap+fill) (inch)	Difference[1] (inch)
C-2-1	1.11	3.88	1.03	3.01	1.00	3.13	7.88	4.75
C-2-2	0.95	3.34	0.60	1.77	0.87	2.42	5.98	3.56
C-2-3	1.66	5.83	0.45	1.35	0.79	2.91	7.96	5.06
C-2-4	1.09	3.81	0.60	1.77	0.87	2.56	6.45	3.89
C-2-5	1.13	3.94	0.54	1.62	0.87	2.54	6.43	3.90
C-2-6	1.36	4.77	0.97	2.94	1.01	3.35	8.73	5.38
C-2-7	1.42	4.98	0.78	2.38	1.00	3.20	8.36	5.15
C-2-8	1.70	5.95	0.77	2.34	1.00	3.47	9.29	5.82
C-2-9	2.93	10.24	1.26	3.82	0.96	5.15	15.03	9.88
C-2-10	0.98	3.44	5.30	15.32	1.17	7.45	19.93	12.47
C-2-11	0.80	2.79	7.83	20.06	1.00	9.63	23.84	14.21
C-2-12	0.81	2.83	5.40	15.95	1.41	7.61	20.18	12.57
C-2-13	0.94	3.28	11.50	30.17	0.42	12.86	33.87	21.01
C-2-14	0.26	0.92	11.56	30.36	0.00	11.82	31.29	19.47
C-2-15	1.30	4.54	4.45	13.28	1.01	6.76	18.84	12.08
C-2-16	0.88	3.07	7.84	24.43	0.00	8.71	27.50	18.79
C-2-17	0.39	1.37	7.85	24.15	0.00	8.24	25.53	17.29
C-2-18	0.53	1.85	5.98	18.60	0.42	6.93	20.88	13.95
C-2-19	0.54	1.87	6.64	20.88	0.51	7.69	23.26	15.57
C-2-20	0.54	1.89	5.78	17.98	0.42	6.75	20.30	13.55
C-2-21	0.27	0.93	8.21	25.14	0.00	8.47	26.07	17.60

Notes:

[1] Difference in total settlements between two loading cases considered.

CPT - cone penetration test

are expected to occur over a very long period of time (up to 400 years) after load application because of the depth and thickness of the Old Bay Mud layer.

4.5.4 Static Slope Stability

The slope stability analyses were performed using the program PC-STABL-5M (Achilleos, 1988). The program was used to obtain factors of safety against sliding failure on different sections at IR Site 2 and the Additional Investigation Area between IR Sites 1 and 2 (Appendix M). The modeling was based on two-dimensional conventional limit equilibrium analyses.

A detailed discussion of the static slope stability analysis performed, and results are presented in Section 4.6.8 under the subheading, Seismic Slope Stability.

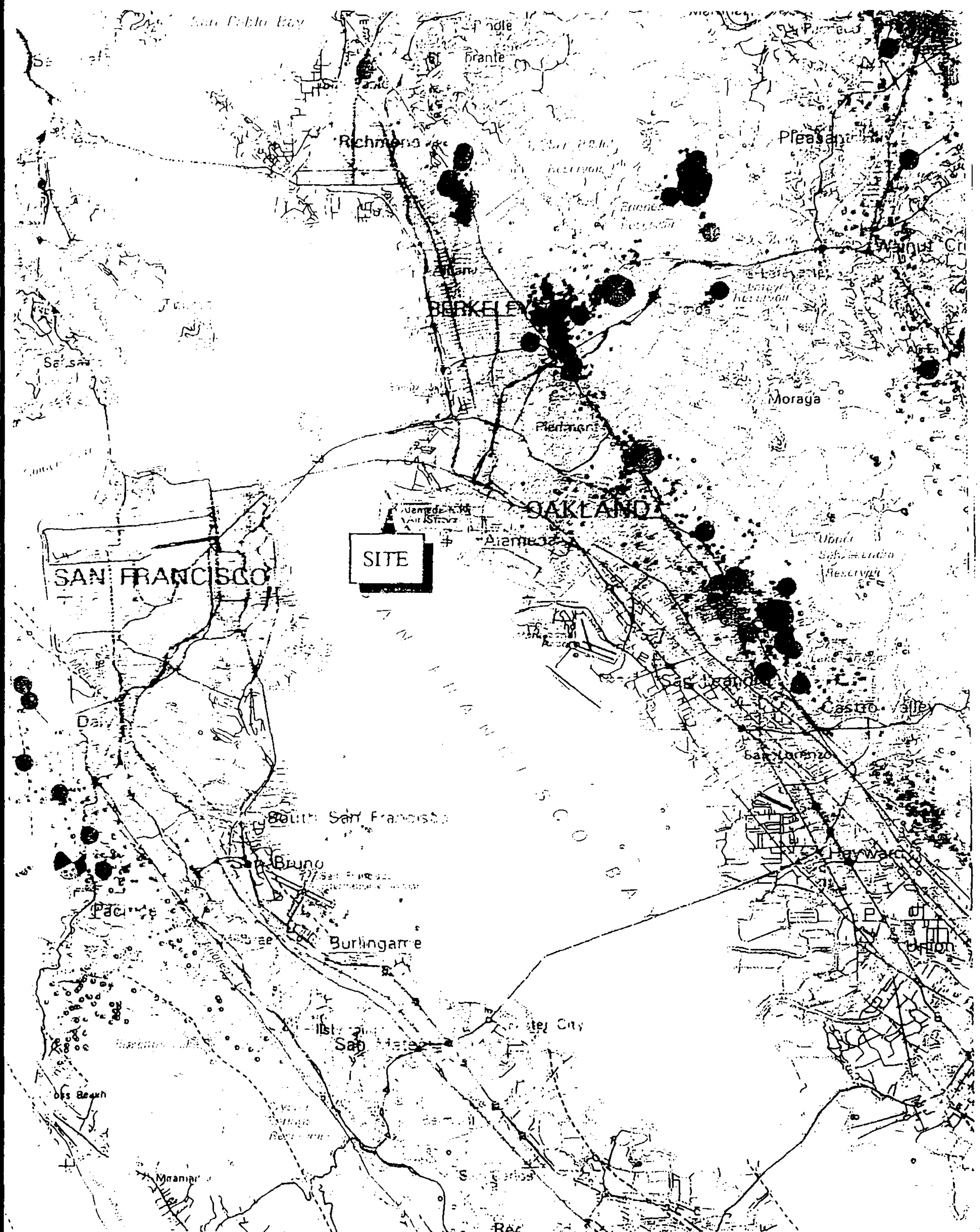
4.6 SEISMIC HAZARD EVALUATION

The seismic hazard evaluation consisted of obtaining site-specific in situ strength properties of soils and other geotechnical characteristics, gathering site-related information on seismicity and faults, determining the site design earthquake ground motions, and performing an engineering assessment of seismic hazards. The seismic hazards evaluated at IR Site 2 and the area between IR Sites 1 and 2 include ground surface fault rupture potential, strong ground shaking, liquefaction (liquefaction-induced settlements and lateral spreading), and slope instability.

Analyses were performed by HAI to evaluate strong ground motion, liquefaction potential and deformations, and seismic (pseudo-static as well as static post-liquefaction) factors of safety and lateral displacements for slope stability. In situ strength of soils and other geotechnical characteristics were discussed in earlier sections. The seismic hazard evaluation performed is summarized in this section.

4.6.1 Seismicity

The site area is located within the seismically active San Francisco Bay region. Figure 4-10 (Walter et al., 1998) shows the locations of earthquakes that occurred between 1967 and 1993. Although the earthquakes on the map cover a short-time interval, they are considered representative of the longer seismic record. The map clearly shows that earthquakes are most abundant along the western edge of the East Bay Hills and along the coastal hills west of the bay. These earthquake concentrations are associated with the Hayward Fault and the San Andreas Fault. The map shows only a few rare, very small magnitude (magnitude less than 3) events scattered widely throughout the region between the faults. The small magnitudes and lack of seismic alignments or clustering suggest that seismicity within the bay is characterized by low magnitude background strain release rather than primary fault tectonics (Olson and Zoback, 1998).



SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-10
SEISMICITY MAP

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

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DCN: FWSD-RAC-03-2899
CTO: #0054
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The largest historical earthquakes in the area are listed on Table 4-7. This list was compiled from published literature. The most notable earthquakes are perhaps the 1906 San Francisco earthquake, which occurred on the San Andreas Fault, and the 1868 earthquake, which occurred on the Hayward Fault. An earthquake in 1836, long considered to have occurred on the Hayward Fault, is now thought to be the 1838 event that occurred near the San Andreas Fault, east of the Monterey Bay region (Topozada and Borchardt, 1998). More recently, the 1989 magnitude 7.1 Loma Prieta earthquake shook the project area. This earthquake occurred about 57 miles from the site, but still caused extensive damage in the site area, due primarily to liquefaction. A more detailed discussion of the strong ground shaking and damage that occurred in the site area during the Loma Prieta earthquake is provided in Section 4.6.3.

In addition to the literature search data summarized in Table 4-7, historical seismicity data were also compiled using the computer program, EQSEARCH (Blake, 2000). The database for EQSEARCH comprises earthquakes occurring between 1800 and 2001. An area within a 63-mile (100-kilometer) radius of the site was searched for all earthquakes of magnitude 4 and larger. The search produced a list of 357 earthquakes. The closest event was about 4 miles from the site; this was a magnitude 5.0 event that occurred on May 15, 1851. The magnitude and location of this event must be regarded with skepticism because there was no known ground rupture and no seismographs at that time. For older earthquakes, the Modified Mercalli Intensity (MMI) scale quantifies damage. The abridged version of the scale is provided in Table 4-8. The MMI for the 1851 event was VIII. The largest magnitude on the list is the 1906 San Francisco event with a magnitude estimated at 8.25 and a MMI of X. The search lists 14 events in the region with magnitudes of 6 or larger and three events of magnitude 7.0 or larger. One of these magnitude 7 events was the 1989 Loma Prieta earthquake.

One of the magnitude 6+ events on the EQSEARCH list is the 1836 event, which is listed as occurring within 7 miles of the site. However, as discussed above, this event is now believed to have been confused for the 1838 event (Toppozada and Borchardt, 1998).

The peak ground motion (horizontal) at the site from the EQSEARCH analysis is approximately 0.40g [acceleration due to gravity, 32.2 feet per second squared (ft/sec^2)]. This acceleration was obtained by assuming a hard rock site using the Borzorgnia et al. (1999) attenuation relationship.

The probability of earthquake occurrence in the San Francisco Bay region has been analyzed by a group of local geologists and seismologists [U.S. Geological Survey (USGS), 1999]. This group of scientists and their investigation is referred to as the 1999 Working Group (WG99). The area of the investigation includes the area from Healdsberg on the north to Salinas on the south. In summary, the WG99 postulated that there is a 70 percent probability of at least one magnitude 6.7 or greater earthquake before the year 2030 within the San Francisco Bay region. They found that the Hayward-Rodgers Creek, San Andreas, and Calaveras Fault systems have the highest probabilities of generating earthquakes within the 30-year time window. The probability of at

TABLE 4-7

BAY AREA EARTHQUAKES HAVING MAGNITUDE > 5.0
(Prepared by HAI)

Earthquake and Year	Reported Magnitude	References
San Francisco, 1838	M 7.0 M_S 7.0 M 7.2	Toppozada et al., 1981 Shedlock et al., 1980 Tuttle and Sykes, 1992
Calaveras-Dublin, 1861	M 5.6 M 5.3	Toppozada et al., 1981 Shedlock et al., 1980
Watsonville, 1864	M 5.9	Toppozada et al., 1981
South Bay Region, 1864	M 5.7	Toppozada et al., 1981
South Bay Region, 1864	M 5.3	Toppozada et al., 1981
Santa Cruz Mountains, 1865	M 6.3 M 6.5	Toppozada et al., 1981 Tuttle and Sykes, 1992
San Juan Bautista, 1865	M 5.5	Toppozada et al., 1981
Gilroy, 1866	M 5.4	Toppozada et al., 1981
Hayward, 1868	M 6.8 M_S 6.7 M 7.2	Toppozada et al., 1981 Shedlock et al., 1980 Toppozada, 1992
Santa Cruz Mountains, 1870	M 5.8	Toppozada et al., 1981
Hayward, 1870	M 5.3	Toppozada et al., 1981
Santa Cruz Mountains, 1884	M 5.9	Toppozada et al., 1981
Antioch-Collinsville, 1889	M 6.0	Toppozada et al., 1981
Hayward, 1889	M 5.2	Toppozada et al., 1981
Pajaro River, 1890	M 6.0 M 6.3	Toppozada et al., 1981 Tuttle and Sykes, 1992
San Jose, 1891	M 5.5	Toppozada et al. 1981
Napa, 1891	M 5.5	Toppozada et al. 1981
Vaca-Winters, 1892	M 6.4 M_L 6.75	Toppozada et al., 1981 Wong, 1984
Vaca-Winters, 1892	M 6.2 M_L 6.25	Toppozada et al., 1981 Wong, 1984
Vaca-Winters, 1892	M 5.5	Toppozada et al., 1981
Santa Rosa, 1893	M 5.1	Toppozada et al., 1981
Gilroy, 1897	M 6.2	Toppozada et al., 1981
Mare Island, 1898	M 6.2 M 6.5 M 6.6	Toppozada et al., 1981 Goter, 1988 Toppozada, 1992
San Francisco, 1899	M 5.4	Toppozada et al., 1981
Morgan Hill, 1899	M 5.8	Toppozada et al., 1981
Solano, 1902	M 5.5	Toppozada and Parke, 1982
Santa Clara, 1903	M 5.5	Toppozada and Parke, 1982

TABLE 4-7

BAY AREA EARTHQUAKES HAVING MAGNITUDE > 5.0
(Prepared by HAI)

Earthquake and Year	Reported Magnitude	References
San Francisco, 1906	M 7.8 M 8.3 M_S 7.8 M_S 8.2	Toppozada et al., 1981 Goter, 1988 Abe and Noguchi, 1983 Shedlock et al., 1980
Santa Cruz, 1910	M 5.5	Toppozada and Parke, 1982
Coyote, 1911	M 6.6	Toppozada and Parke, 1982
Santa Clara, 1914	M 5.5	Toppozada and Parke, 1982
Concord, 1955	M_L 5.4	Tocher, 1959
San Francisco, 1957	M_L 5.3 M_L 5.2	Tocher, 1959
Watsonville, 1963	M_S 5.4 M_L 5.4 M_W 5.2	Evans and McEvilly, 1982 Utsu, 1969
Corralitos, 1964	M_S 5.0 M_L 5.1	McEvilly, 1966
Santa Rosa, 1969	M_L 5.6 M_W 5.4	Cloud et al., 1970 Scott, 1970
Santa Rosa, 1969	M_L 5.6	Cloud et al., 1970 Scott, 1970
Bear Valley, 1972	M_L 5.1 M_W 5.2	Ellsworth, 1975 Johnson and McEvilly, 1974
Coyote Lake, 1979	M_S 5.7 M_L 5.9 M_W 5.8	Bouchon, 1982 Uhrhammer, 1980 King et al., 1981 NEIC
Greenville, 1980	M_L 5.6 M_S 5.9 M_L 5.5 M_W 5.8	Hart, 1988 Shedlock et al., 1980 Bolt et al., 1981
Morgan Hill, 1984	M_S 6.1 M_L 6.2 M_W 6.2	Hoose, 1987 NEIC
Mt. Lewis, 1986	M_S 5.5 M_L 5.7 M_W 5.6	NEIC/U.C. Berkeley; Bolt and Uhrhammer, 1986
Alum Rock, 1988	M 5.1	Du and Aydin, 1992
Loma Prieta, 1989	M_S 7.1 M_W 6.9	USGS, 1989

Notes:

HAI – Hushmand Associates, Inc.
M – magnitude
M_L – local magnitude (also often referred to as Richter magnitude scale)
M_s – surface wave magnitude scale
Mw – moment magnitude scale
NEIC – National Earthquake Information Center
USGS – United States Geological Survey

TABLE 4-8

MODIFIED MERCALLI INTENSITY SCALE (1931-ABRIDGED)
(Prepared by HAI)

MMI Scale	Definition
I	Not felt except by a very few under especially favorable circumstances.
II	Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.
III	Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing of truck. Duration estimated.
IV	During the day, felt indoors by many, outdoors by few. At night, some awakened. Dishes, windows, doors disturbed; walls made cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.
V	Felt by nearly everyone; many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop.
VI	Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.
VII	Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motor cars.
VIII	Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons disturbed while driving motor cars.
IX	Damage considerable in specially designed structures; well designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.
X	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks.
XI	Few, if any (masonry), structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipe lines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.
XII	Damage total. Waves seen on ground surfaces. Lines of sight and level distorted. Objects thrown upward into the air.

Notes: Source: Wood and Neuman, 1931

HAI – Hushmand Associates, Inc.
MMI – Modified Mercalli Intensity

least one smaller (magnitude 6.0 to 6.7) earthquake in the San Francisco Bay region before 2030 is estimated to be at least 80 percent.

The Hayward-Rodgers Creek Fault system has the highest 30-year probability of 32 percent. The next highest probability is for the San Andreas Fault, whose lower probability of 21 percent reflects both the larger magnitude considered and the 20th-century relaxation of strain due to the occurrence of the 1906 earthquake. The Calaveras Fault has a probability of 18 percent by itself, but when associated with the Concord-Green Valley, Greenville, and Mt. Diablo faults, the probability rises to about 30 percent.

4.6.2 Faults

Figure 4-6 shows a map of regional faults. There are no known faults directly at or in the near vicinity of the subject site (see Figure 4-6). No earthquake fault zones (Alquist-Priolo Zones) have been designated at the site. The nearest active fault is the Hayward Fault, which is about 7 miles (11 kilometers) east of the site (Jennings, 1994). Another nearby active fault is the San Andreas Fault within the hills on the west side of San Francisco Bay at a distance of about 12 miles (19 kilometers). Other major faults in the region comprise the Calaveras Fault system on the east side of the East Bay Hills and the Green Valley and Greenville Fault systems, which are farther to the east. Figure 4-6 shows these and associated faults. Table 4-9 provides information on the seismic sources (faults), which contribute to the seismic hazard at the site. Information on the fault type, geometry (length, width, dip angle, and direction), and slip rate and maximum magnitude are provided. The fault parameters were derived from the recent fault database compiled as part of the seismic hazard evaluation model developed for the state of California by the California Division of Mines and Geology (CDMG) and USGS (Petersen et al., 1996). The fault database information was also checked against the recent information developed for seismic retrofit design of the west span of the SFOBB (Geomatrix, 1995) and design and construction of the new east span of the SFOBB (Fugro-EMI, 2001a). Because the Hayward and San Andreas faults appear to control seismic design, they are described in more detail below.

San Andreas Fault. The San Andreas Fault extends throughout much of the length of California and is the principal boundary fault between the Pacific and North American plates. The San Andreas Fault extends from at least the Salton Sea in the southernmost area of California to the Cape Mendocino area, a distance of almost 1,100 kilometers.

The fault has been the source of two great earthquakes in historical time, the magnitude 7.9 moment magnitude scale (M_W) event of 1857 in central California and the magnitude 7.9 (M_W) event of 1906 in northern California. Based on these historical events, the maximum earthquake (M_W) for the San Andreas Fault is 8.0. The CDMG (Petersen et al., 1996) estimated a maximum earthquake magnitude of 7.9 for the segment of the fault closest to the site.

TABLE 4-9

SEISMIC PARAMETERS OF SIGNIFICANT FAULTS WITHIN 100 KILOMETERS OF SITE (Prepared by HAI)

Fault Name and Geometry [1]	Distance from Site (km)	Length (km)	Slip Rate (mm/yr)	Down Dip Width (km)	Maximum Earthquake	Recurrence Interval (years)
Hayward (north) (rl-ss)	11.2	43±4	9±1	12	6.9	167
Hayward (total length) (rl-ss)	11.2	86±9	9±1	12	7.1	167
Hayward (south) (rl-ss)	18.5	43±4	9±1	12	6.9	167
San Andreas (1906) (rl-ss)	18.7	470±47	24±3	12	7.9	210
San Andreas (peninsula) (rl-ss)	18.7	88±9	17±3	14	7.1	400
San Andreas (north coast) (rl-ss)	23.4	322±32	24±3	12	7.6	N/A
San Gregorio (rl-ss)	24.1	80±8	3±2	12	7.0	411
Calaveras (northern segment) (rl-ss)	27.4	52±5	6±2	13	6.8	146
Concord - Green Valley (rl-ss)	32.9	66±7	6±3	12	6.9	176
Rodgers Creek (rl-ss)	33.7	63±6	9±2	10	7.0	222
Greenville (rl-ss)	40.1	73±7	2±1	11	6.9	521

Notes:

(1) Fault Type/Geometry Definitions: (rl-ss) Right Lateral, Strike Slip Fault with 90-degree Dip Angle
HAI – Hushmand Associates, Inc.
km – kilometer
mm/yr – millimeters per year
N/A – not applicable
rl-ss – right lateral, strike slip

Hayward Fault. The Hayward Fault was the source of a magnitude 6.8 earthquake in 1868 (Toppozada et al., 1981). The magnitude of the earthquake was estimated from intensity data. The Hayward Fault has a mapped length of more than 100 kilometers from the Mt. Misery area southeast of San Jose to Point Pinole north of Richmond (see Figure 4-6). The fault approaches the Calaveras Fault on the south end, but the interconnection between them is very complex. The fault extends northerly into San Pablo Bay. Faults on the north side of San Pablo Bay, such as the Rodgers Creek, Tolay, and Burdell Mountain Faults extend southerly into San Pablo Bay. The relationships of these faults to the Hayward Fault are uncertain, but there are enough dissimilarities and discontinuities to suggest that the features are separate and most seismic hazards analyses consider them to be discrete seismic sources.

The maximum earthquake for the Hayward Fault is uncertain. Estimates based on length-magnitude relationships (Wells and Coppersmith, 1994) indicate a magnitude in excess of 7.0. Various studies have postulated different rupture lengths based on assumed segment lengths. The most recent probabilistic seismic assessment for the State of California (Petersen et al., 1996) and the fault database model developed as part of that study, used a maximum magnitude of 7.1 for the Hayward Fault. Many seismic hazard analyses used values of about 7.25 prior to the development of the 1996 State of California fault database. The maximum value used for the probabilistic seismic hazard analysis of the new east span of the San Francisco Bay Bridge was 7.2, whereas the California Department of Transportation (Caltrans) (Mualchin, 1996) estimated a magnitude 7.5. In the deterministic seismic ground motion evaluations performed in this study for the Alameda Point site, the maximum magnitude of 7.1, consistent with the value selected by the CDMG and USGS (Petersen et al., 1996), was used for the Hayward Fault.

4.6.3 Previous Seismic Field Experience at Alameda Point

The largest, well-recorded seismic event in the San Francisco Bay area is the 1989 surface magnitude scale (M_S) 7.1 Loma Prieta earthquake, which occurred approximately 60 miles south of the cities of San Francisco and Oakland. The Alameda Point site was shaken moderately. The MMI scale of 1931 (Wood and Neuman, 1931) assigned to the site area was VII. However, MMI assigned along SFOBB, the Nimitz Freeway and Cypress Street viaduct (I-880), and Oakland Mole, located less than 3 miles from the site, was IX. The closest strong motion seismic station located at Alameda Point at Hangar 23, less than 1 mile from IR Site 2, recorded a maximum peak horizontal ground acceleration (PHGA) of about 0.21g. The seismic station at Oakland Outer Harbor Wharf Station, which is located on a concrete wharf structure about 1.6 miles from the site, recorded a maximum ground surface acceleration of approximately 0.3g. At another station (located on the first floor of a two-story office building) in Oakland, less than about 4 miles from the site, the largest PHGA recorded was 0.26g. The station at Treasure Island located within a one-story building at the former Naval Base Fire Station facility, approximately 3 miles from the site, recorded a maximum PHGA of 0.16g.

During the Loma Prieta Earthquake, the Bay Bridge and the Cypress Street viaduct, less than 3 miles from the site, collapsed, and the damage at Port of Oakland, less than 2 miles from the site, was extensive. The damage at Port of Oakland occurred at several of the port container terminals due to liquefaction of the loose, hydraulically placed sand fills. Liquefaction resulted in sand boils, ground settlements, lateral spreading of perimeter dikes and backland fills, failure of the pile foundations, and damage to crane structures.

Large areas of the sand fill liquefied during the earthquake (Earthquake Spectra, 1990) at Alameda Point, which is constructed on hydraulically placed sand fill and is contained by rock seawalls. Both runways and two taxiways had major surface defects and were closed to normal aircraft operations. Liquefaction resulted in sand boils, damage to pavements, and longitudinal and transverse cracks varying in width from hairline to 4 inches, with a vertical differential of 0.5 to 2 inches. The cracks extended down 3 to 4.5 feet deep. No survey information on the earthquake-induced ground surface deformations at the Alameda Point site is available.

4.6.4 Ground Surface Fault Rupture Hazard

As discussed in Section 4.6.2, there are no known faults located directly at or in the very near vicinity of the subject site. No earthquake fault zones (Alquist-Priolo Zones) have been designated at the site, so there is little or no potential for surface ground displacement due to fault surface rupture.

4.6.5 Ground Response Analyses

A deterministic evaluation of seismic shaking hazard at the IR Site 2 and the area between IR Sites 1 and 2 was conducted to estimate the earthquake shaking levels due to the Maximum Earthquakes [also defined as Maximum Credible Earthquake (MCE) in Title 27 California Code of Regulations (CCR)] on seismic sources that could result in potentially damaging ground motions at the site. The deterministic evaluation provides an estimate of the site design ground motion for the maximum earthquake on fault(s) contributing most to the site seismicity without any reference to the probability associated with the earthquake occurrence. The design accelerations were derived for two locations on the site. One at the middle of the site western boundary along the San Francisco Bay shoreline (Point 1) and the other at the middle of the site southern boundary (Point 2) (see Figure 2-2). The ground motions at these two locations were found to be similar. The analyses were performed using:

- The most recent information on faulting and seismicity of northern California
- Attenuation equations developed after the January 17, 1994, Northridge earthquake
- CDMG Special Publication 117 (1997) and CDMG Note 42
- Latest developments in evaluation of near-fault effects
  (for example, directivity effects)

The site design PHGAs for the base rock outcrop were evaluated assuming a "rock/stiff soil" site condition. The base rock motions at a depth of approximately 400 feet bgs were used to estimate the site design ground surface motions. Five empirical attenuation relationships for rock/stiff soil sites were selected for this analysis. Attenuation relationships describe the relation of ground motion levels with earthquake magnitude and distance (distance between site and earthquake rupture). These relationships are used to describe the statistical variation of response spectral accelerations at specific structural periods of vibration and damping ratios (including PHGA) with earthquake magnitude and distance, and to incorporate the local geologic conditions and the near-source effects. The four selected relationships are listed below:

- Boore et al. (1997) for "rock (620)" (rock with a shear-wave velocity of 620 meters/second)
- Bozorgnia et al. (1999) for "hard rock"
- Idriss (1994) for "rock/stiff soil"
- Abrahamson and Silva (1997) for "rock"
- Sadigh et al. (1997) for "rock"

The PHGAs estimated from the above attenuation relationships and the mean values are presented in Table 4-10. The deterministic analyses resulted in the site average PHGA estimates of approximately 0.32g and 0.33g at the rock surface due to the Maximum Earthquakes of magnitude 7.9 and magnitude 7.1 on the San Andreas and Hayward faults, respectively (see Table 4-10). For estimated peak rock accelerations in Table 4-10, liquefaction and slope instability hazards at the site are more influenced by the magnitude 7.9 earthquake on the San Andreas Fault, rather than the magnitude 7.1 earthquake on the closer Hayward Fault. The larger magnitude earthquake at a farther distance from the site results in a longer duration of shaking and thus more severe liquefaction and slope instability hazards.

The historical seismicity data suggests that the site might have experienced a maximum rock acceleration of up to approximately 0.4g in the past 200 years (due to the 1906 San Francisco earthquake on San Andreas Fault, see Section 4.6.1). The estimated rock acceleration of 0.4g at the project site due to the 1906 San Francisco earthquake on the San Andreas Fault is not a recorded historical acceleration. However, it is common practice in the industry to use reasonable estimates of the site historical rock acceleration based on the estimated earthquake magnitude, site epicentral distance, and recent attenuation relations. Table 4-10 also shows that some of the attenuation equations result in a median peak horizontal ground acceleration of approximately 0.35g or higher at the site. Therefore, based on the above facts and due to uncertainties associated with any seismic hazard analysis, a site design maximum rock acceleration of 0.4g (rounded to the nearest tenth higher than the estimated acceleration) was assumed for seismic evaluations.

TABLE 4-10

SITE PEAK HORIZONTAL GROUND ACCELERATIONS

Point	Faults	Earthquake Magnitude M	Distance (km) D	Peak Horizontal Ground Acceleration (g)					Mean Value (g)
				Bozorgnia et al. (1999)	Abrahamson and Silva (1997)	Sadigh et al. (1997)	Idriss (1994)	Boore et al. (1997)	
1	Hayward	7.1	11.2	0.37	0.35	0.36	0.32	0.25	0.33
	San Andreas	7.9	18.7	0.37	0.30	0.34	0.31	0.27	0.32
2	Hayward	7.1	11.4	0.36	0.34	0.35	0.31	0.24	0.32
	San Andreas	7.9	18.6	0.37	0.30	0.34	0.27	0.31	0.32

Notes:

g – acceleration due to gravity 32.2 feet per second squared (ft/sec^2)
km – kilometer

Table 4-11 summarizes the best estimated design ground motion parameters (earthquake magnitude, peak horizontal acceleration, mean period, strong ground shaking duration, and fault to site distance) for the input rock motions estimated for the faults controlling the seismic hazard at the site (mainly, the Hayward and San Andreas faults). The selected earthquake parameters provide a conservative estimate of the site design ground motion based on a magnitude 7.9 event on the San Andreas Fault with a relatively long shaking duration and a maximum PHGA of 0.40g, which is greater than those computed due to the MCE on the Hayward or San Andreas faults (0.33g and 0.32g, respectively).

Section 4.6.5.2 also provides a comparison of site acceleration response spectra computed for the MCE on the Hayward or San Andreas faults. The rock motion mean period ($T_{m\text{-}EQ}$) and strong ground shaking duration ($D_{5\text{-}95}$) were estimated from plots in Figure 4-11.

4.6.5.1 Local Soil Deposit Effects on Ground Motion

The site was characterized according to the *NEHRP Recommended Provisions* [Building Seismic Safety Council (BSSC), 1997] as a Class F soil site based on its predominant stratigraphy and results of the field subsurface investigation, including shear-wave velocity (V_S) measurements using a seismic cone (see Appendix B), and laboratory test results. Class F is defined as soils requiring site-specific evaluations. These include soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible, weakly cemented soils. The average shear-wave velocity for the top 100 feet of the site soils was measured to be in the range of 590 to 890 ft/sec, while in the upper 60 feet, it varied from approximately 490 to 660 ft/sec. These measurements are consistent with the velocity profile measured by the USGS at the Alameda Point seismic station located at Hangar 23 of the former naval base (Fumal, 1991), which estimated an average shear-wave velocity of 705 ft/sec for the top 30 meters (approximately 100 feet) of soils. The potential amplification of the bedrock motions by the local soil deposits was estimated using the following methods:

a) Preliminary estimate of the site PHGA acceleration was derived from an empirical relationship (Figure 4-12) developed by Idriss (1990) for soft soil sites. The recommended median relation in the figure indicates that the site peak bedrock acceleration of 0.40g does not significantly amplify or attenuate at the ground surface. Therefore, the site PHGA is estimated to be approximately equal to 0.40g based on this approach. This approach provides a preliminary estimate of the site design ground motion based on the relations developed using recorded rock and soil surface earthquake ground motions and site response analytical methods.

b) More realistic site-specific and at the same time conservative estimates of the site seismic response are provided by one-dimensional dynamic response analyses. At the Alameda Point site, bedrock is encountered at a great depth (elevation approximately -400 feet msl), and the ground response is influenced by the deep deposits of soil sediments. Therefore, free-field site response studies such as one-dimensional wave propagation analyses, which include the effects of the site soils layering and dynamic properties are

TABLE 4-11

SELECTED DESIGN ROCK MOTION CHARACTERISTICS

Magnitude	Distance (km)	Peak Horizontal Acceleration (g)	Duration (sec)	Mean Period (sec)
7.9	~ 19	0.4	35	0.55

Notes:

g – acceleration due to gravity 32.2 feet per second squared (ft/sec^2)
km – kilometer
sec – seconds

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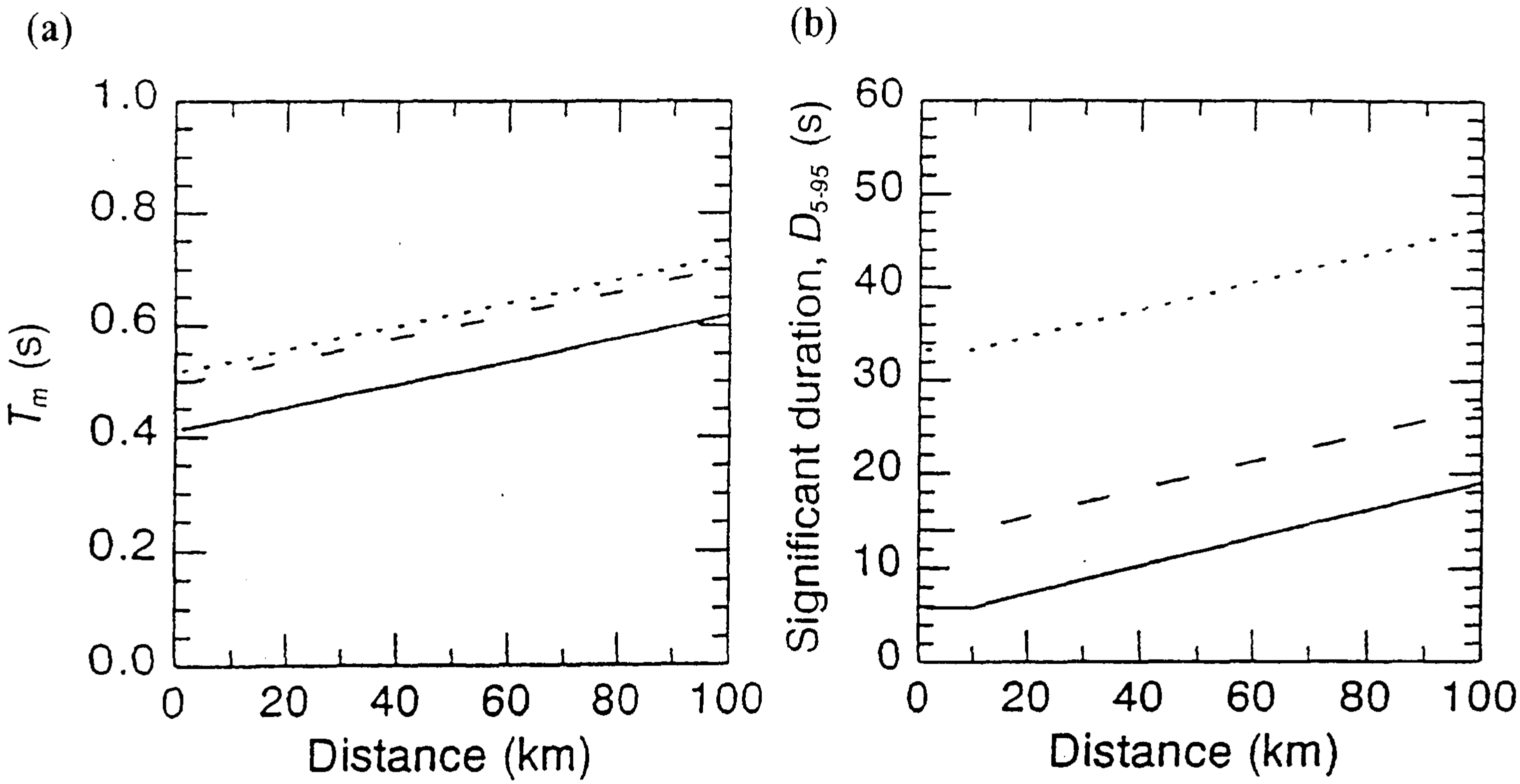

(a) Frequency Content, T_m (Rathje et al. 1998)

(b) Duration, $D_{5\text{-}95}$ (Abrahamson and Silva 1996)

LEGEND

- - - - - - - - - - - $M_W = 8.0$

— — — — $M_W = 7.0$

———————— $M_W = 6.0$

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4–11
SIMPLIFIED CHARACTERIZATION
OF EARTHQUAKE ROCK MOTIONS

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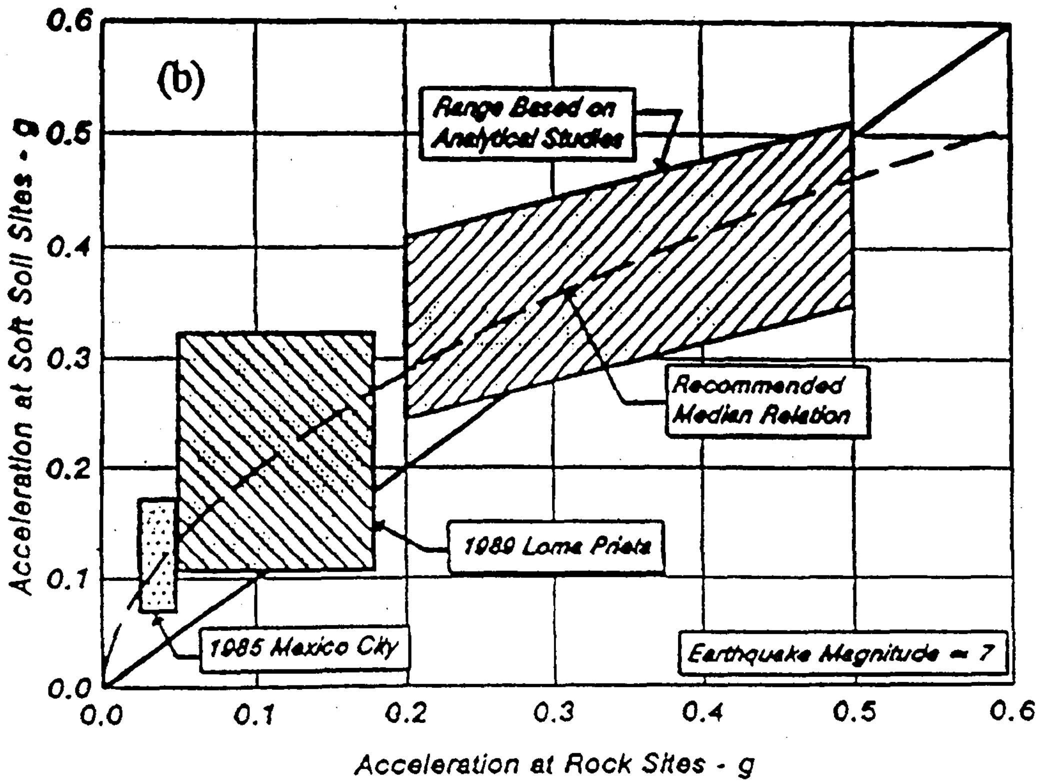

(after Idriss 1990)

g = Gravity Force

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-12
RELATIONSHIP BETWEEN MAXIMUM ACCELERATION ON ROCK AND OTHER LOCAL SITE CONDITIONS

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pertinent for evaluation of the near-surface ground motions. The site response analysis requires developing representative input rock earthquake motions to be used at the base of the soil profile modeling the site subsurface conditions. Details of the approach for the one-dimensional response analyses, including selection of the input rock motions and the site dynamic soil properties, are provided in Section 4.6.5.2.

4.6.5.2 One-Dimensional Site Response Analyses

Dynamic one-dimensional response analyses were performed for three 410-foot-thick "infinitely long" layered soil systems representing the site subsurface conditions at three CPT locations. These are:

a) Profile 1 at CPT Location C-2-6 representing site soils along IR Site 2 southern boundary

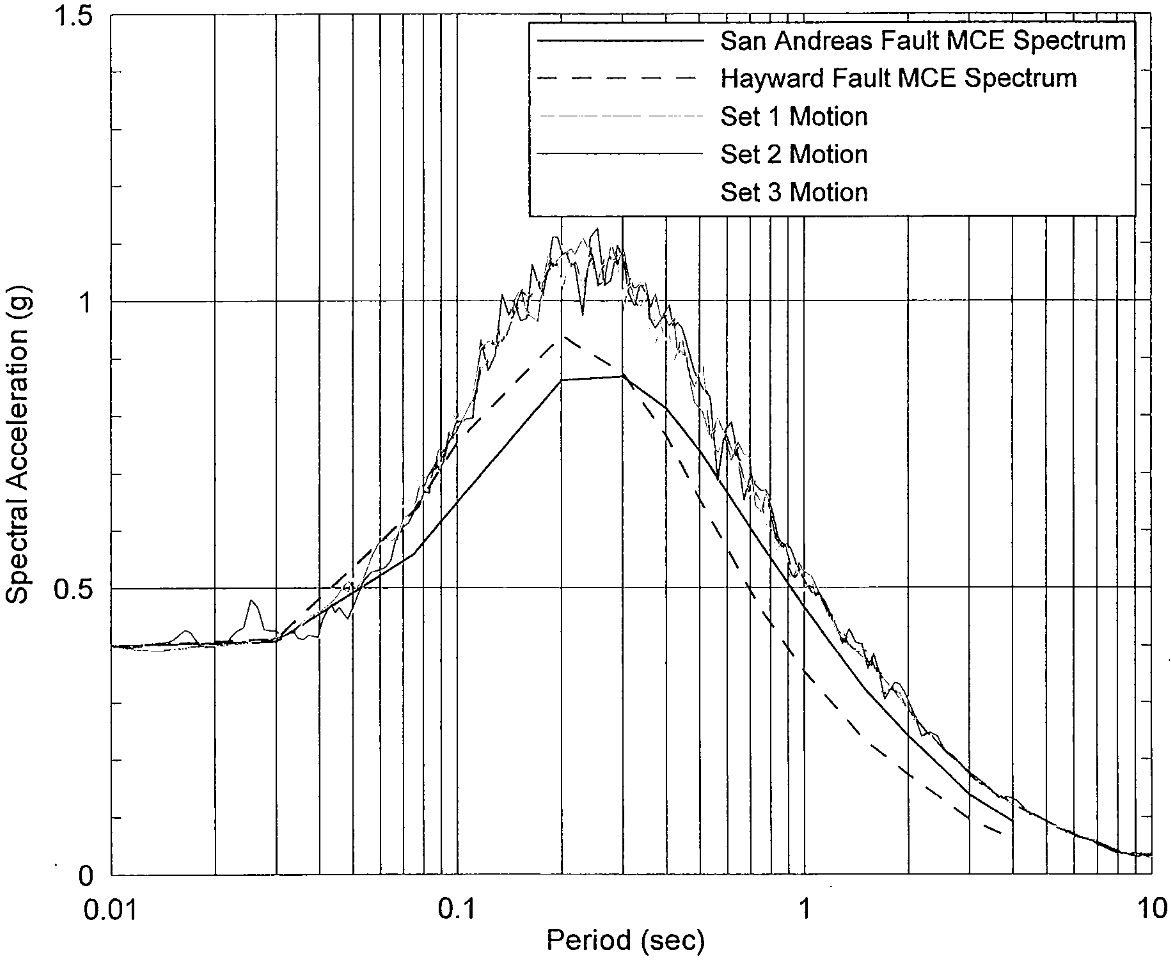
b) Profile 2 at CPT Location C-2-13 representing site soils along IR Site 2 western boundary

c) Profile 3 at CPT Location C-2-19 representing site soils within the area between IR Sites 1 and 2

Computations were conducted using the computer program, SHAKE91 (Schnabel et al., 1972; Idriss and Sun, 1991). The program computes the response of a semi-infinite horizontally layered soil deposit overlying a uniform half-space subjected to vertically propagating shear waves. The analysis is done in the frequency domain, and therefore, for any set of properties, it is a linear analysis. An iterative procedure is used to account for the nonlinear behavior of the soils. The object motions (input motions that are considered to be known) were specified at the top of the bedrock underlying the site. The steps involved in developing the object motions (representative input rock acceleration time histories for the site) include:

<u>Step 1 - Estimate Site Design Acceleration Response Spectrum</u>

Deterministic evaluation of the site design response spectra were performed using the same input parameters and attenuation relations used to estimate the site PHGA in order to derive the site design response spectrum. The attenuation relationships developed by Sadigh et al. (1997), Abrahamson and Silva (1997), Idriss (1994), and Boore et al. (1997) were used to estimate 5-percent damped spectral accelerations for the MCE on the San Andreas Fault (the design event). These relationships, which have been developed based on available recorded data and are typical for a spectral damping of 5 percent, provide estimates of the median spectral ordinates and a measure of the dispersion about the median (Idriss, 1993). The median response spectra for the above attenuation relationships were averaged to derive the site design response spectrum as shown in Figure 4-13. Figure 4-13 illustrates the 5-percent damped median site design response spectrum developed for a magnitude 7.9 earthquake on the San Andreas Fault (MCE associated with the fault) at a distance of approximately 19 kilometers from the site. The computed site design response spectrum was then scaled up to the PHGA of 0.40g. The figure also illustrates, for comparison, the 5-percent damped median response spectrum for the MCE on the Hayward



Note: Set1, Set2, and Set3 Acceleration Records Were Developed for SFOBB Project

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-13
SELECTED EARTHQUAKE RECORDS RESPONSE SPECTRA vs. SITE DESIGN RESPONSE SPECTRUM

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Fault scaled to the site design PHGA of 0.40g. Note that the San Andreas Fault MCE design response spectrum, after being scaled to the PHGA of 0.40g, results in a more conservative ground motion estimate for periods larger than 0.3 second compared to that estimated from the MCE on the Hayward Fault (magnitude 7.1) at a distance of approximately 11 kilometers from the site. As discussed in Section 4.6.5 (see Figure 4-11 and Table 4-11), the natural period of the most critical potential failure mass in the slope stability analyses for the landfill perimeter slopes was estimated to be on the order of 0.5 second, which is in the range where the San Andreas Fault results in a larger ground motion compared to the Hayward Fault.

Step 2 - Select Representative Site Ground Motions (Horizontal Acceleration Time Histories)

Recently, as part of the SFOBB East Span Seismic Safety project (SFOBB project), a comprehensive and extensive seismic hazard evaluation was performed to develop representative earthquake ground motions for different sites along the SFOBB east span due to seismic events on San Andreas and Hayward faults. The study was performed by a team of internationally known earthquake engineers and seismologists selected by Caltrans. The Caltrans study (Fugro-EMI, 2001a) is relevant to Alameda Point because the easternmost section of the SFOBB near Oakland Mole is only about 1.5 miles north of the landfill and appears to have nearly identical geological/geotechnical conditions. Therefore, it was decided that the acceleration time histories developed for the SFOBB project provide appropriate and conservative representative ground motions for the site response and slope deformation analyses at the landfill site.

It is always preferable to select the site representative ground motions from recorded earthquake acceleration time histories rather than synthetic records. However, there are no empirical time histories that are directly applicable to the magnitude and distance range for the site design earthquake due to an event on the San Andreas Fault. The SFOBB project design ground motions representing earthquakes on the San Andreas Fault were developed using the following two approaches: splicing together empirical time histories and numerical simulations. Given the strong preference for empirical data, two sets of records were developed based on splicing together empirical time histories and one set developed using numerical solutions. The first set (Set 1) was based on the 1940 Imperial Valley earthquake recorded at El Centro. To increase the duration of this record to be appropriate for a magnitude 7.9 earthquake, the recording from the 1979 Imperial Valley earthquake recorded at El Centro Arrary No. 6 was added to the end. The other empirical set (Set 3) was based on the 1992 Landers earthquake recorded at Lucerne, San Bernardino County, California. To increase the duration, the recordings from the CDMG Joshua Tree Station in San Bernardino County, California, were appended to the Lucerne recordings. The numerical simulation (Set 2) was selected from several alternative simulations generated for the SFOBB project. The Set 2 recordings were for a magnitude 8 earthquake on the San Andreas Fault at a distance of 18 kilometers. The selected sets of recordings included the fault rupture directivity effects (records with forward rupture directivity effects). Each set included a fault normal component, a fault parallel component, and a vertical component. The fault normal components, which were the more conservative horizontal ground motions, were selected as input motions in one-dimensional seismic response analyses of the landfill site. Figure 4-13 shows a comparison of the site design response spectrum for the MCE event and the 5-percent damped response spectra of the selected records, scaled to a peak acceleration of 0.40g (site design peak horizontal acceleration at rock surface). Figure 4-14a illustrates time histories of the selected acceleration records. Figures 4-13 and 4-14a illustrate that the selected acceleration time histories have frequency content and durations representative of seismicity and geology of the project site and provide conservative estimates of the site design ground motions.

Five generalized soil type layers overlying the foundation Franciscan Formation bedrock were used to model the subsurface soil conditions in the one-dimensional SHAKE91 analyses. Figure 4-14b illustrates an example of the soil layering model used in SHAKE91 analyses representing the site subsurface conditions along IR Site 2 western boundary (Soil Profile 2 at CPT Location C-2-13 location). The unit weight and shear-wave velocity profiles used in the dynamic site response analyses, summarized in Tables 4-12a, b, and c, were derived from the site-specific field and laboratory test results obtained for IR Site 2 and the area between IR Site 1 and 2 soils during this investigation (generally at depths less than 100 feet), and the data provided for the SFOBB project for the deeper soil layers to the depth of bedrock (Fugro-EMI, 2001a; 2001b). Field exploration including CPT soundings and soil borings were performed at the site to measure in situ penetration resistance and seismic-wave velocities and to recover soil samples for measuring in situ moisture and density. The unit weight and shear-wave velocity of the foundation Franciscan Formation bedrock were assumed to be to 140 pcf and 5,000 feet/sec^2, respectively. Relations used for the site soils to define the reduction of shear modulus ratio (G/G_{max}) and the increase of damping ratio (β) versus shear strain were:

- Type 1: Average modulus and damping relations for loose sand (Seed and Idriss, 1971)
- Type 2: Modulus and damping relations for Young Bay Mud (Idriss, 1990; Pyke, 1995)
- Type 3: Upper bound shear modulus ratio relation for dense sand (Seed and Idriss, 1971); and the lower bound damping ratio relation for dense sand (Idriss, 1990)
- Type 4: Modulus and damping relations for Old Bay Mud (Idriss, 1990; Pyke, 1995)
- Type 5: Modulus (upper range) for clay (Sun et al., 1988) and damping for clay (Idriss, 1990)

The above relations are plotted in Figure 4-14c. Table 4-12a, 4-12b, and 4-12c and Figures 4-14a, 4-14b, and 4-14c summarize the input data for SHAKE91 analyses.

Sufficient site-specific data for waste material properties are not available. The site-specific field exploration performed for this study concentrated on a narrow zone along the site perimeter, which possibly included some waste material.

The waste material was modeled approximately as the upper 20-foot-thick soil layer (fill) with material properties estimated based on the results of field exploration and laboratory testing

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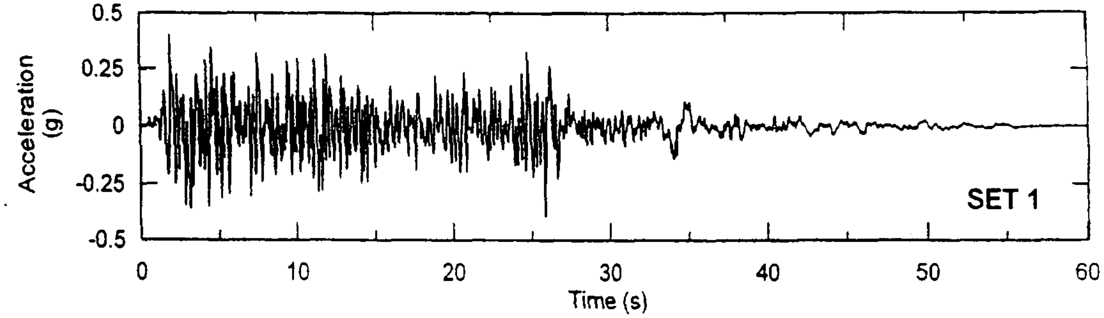
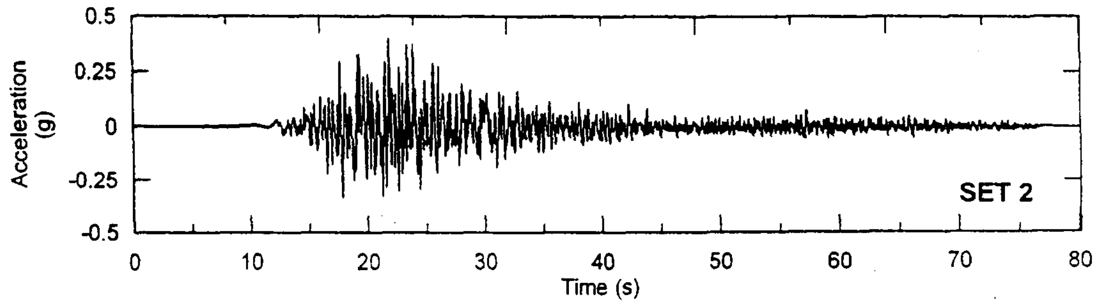
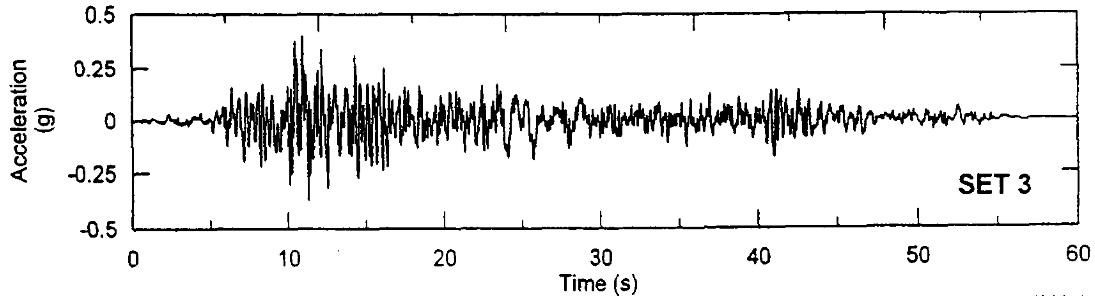
Set 1: 1940 Imperial Valley Earthquake, El Centro Record with 1979 Imperial Valley Earthquake, El Centro Array No. 6 Record added to the end.



Set 2. Numerically Simulated Magnitude 8 Earthquake on the San Andreas Fault at a distance of 18 km.



Set 3: 1992 Landers Earthquake, Lucerne Record with the Joshua Tree Station Record added to the end (both stations located in San Bernardino County).



SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-14a
SELECTED ACCELERATION TIME HISTORIES

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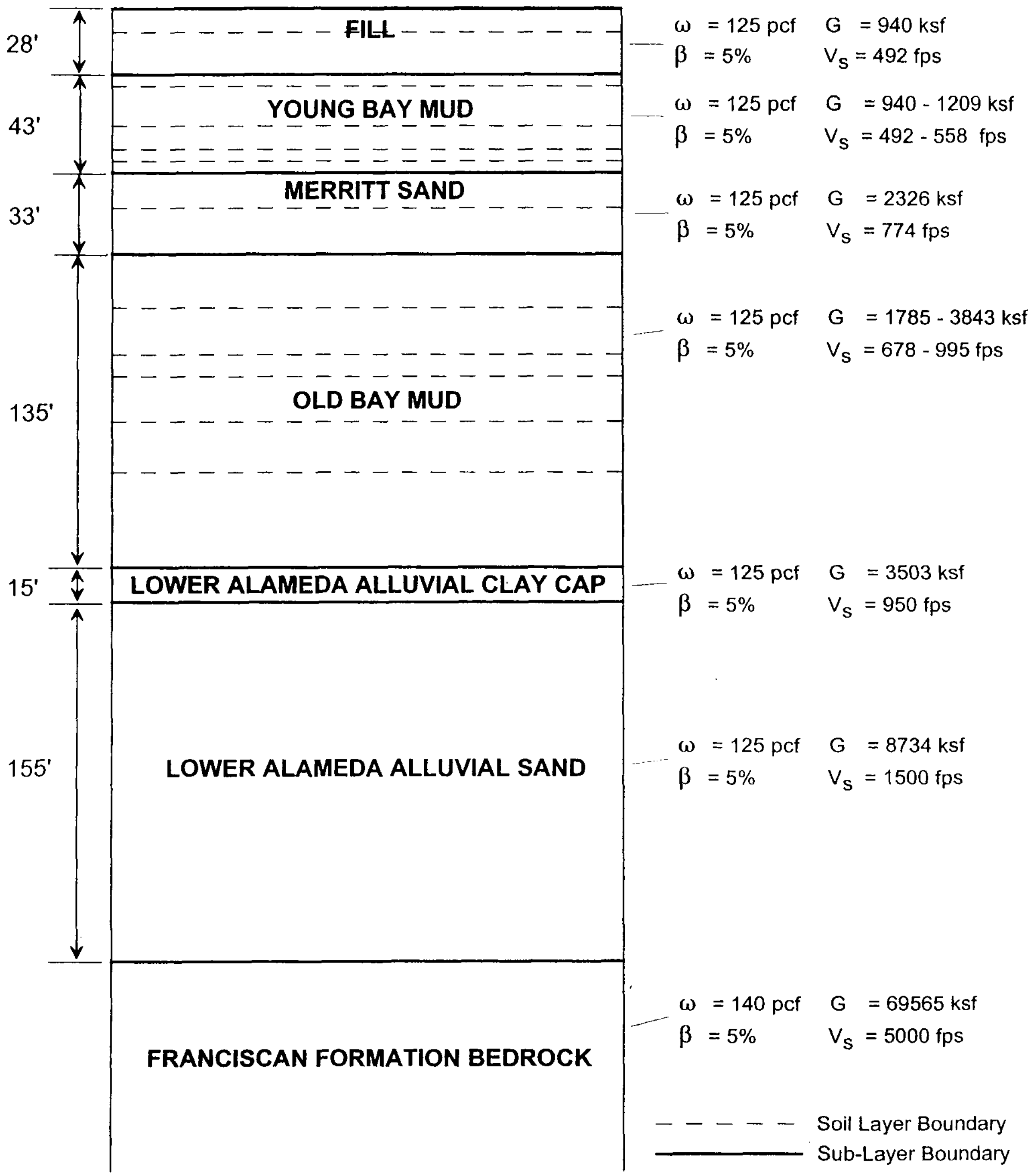

ω = unit weight G = shear modulus
β = damping ratio V_S = shear wave velocity

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-14b
SOIL LAYERS FOR SHAKE91 ANALYSES

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TABLE 4-12a

SHAKE91 ANALYSIS INPUT PARAMETERS FOR SOIL PROFILE 1

Layer No.	Soil Type	Layer Thickness (feet)	Damping	Unit Weight (kcf)	Shear Velocity (ft/sec)
1	1	10.00	0.05	0.130	656.0
2	1	15.00	0.05	0.130	656.0
3	1	19.00	0.05	0.130	656.0
4	2	11.00	0.05	0.110	656.0
5	3	5.00	0.05	0.130	656.0
6	3	20.00	0.05	0.130	1353.0
7	4	10.00	0.05	0.125	1353.0
8	4	15.00	0.05	0.125	688.0
9	4	23.00	0.05	0.125	826.0
10	4	20.00	0.05	0.125	902.0
11	4	9.50	0.05	0.125	678.0
12	4	19.50	0.05	0.125	995.0
13	4	22.00	0.05	0.125	800.0
14	4	41.00	0.05	0.125	895.0
15	5	15.00	0.05	0.125	950.0
16	3	155.00	0.05	0.125	1500.0
17	Base (6)	--	0.01	0.140	5000.0

Notes:

-- – no information available
kcf – kips per cubic foot
ft/sec – feet per second

TABLE 4-12b

SHAKE91 ANALYSIS INPUT PARAMETERS FOR SOIL PROFILE 2

Layer No.	Soil Type	Layer Thickness (feet)	Damping	Unit Weight (kcf)	Shear Velocity (ft/sec)
1	1	10.00	0.05	0.130	492.0
2	1	23.00	0.05	0.130	492.0
3	2	17.00	0.05	0.110	492.0
4	2	10.00	0.05	0.110	492.0
5	2	10.00	0.05	0.110	558.0
6	3	15.00	0.05	0.130	774.0
7	3	20.00	0.05	0.130	774.0
8	4	23.00	0.05	0.125	826.0
9	4	20.00	0.05	0.125	902.0
10	4	9.50	0.05	0.125	678.0
11	4	19.50	0.05	0.125	995.0
12	4	22.00	0.05	0.125	800.0
13	4	41.00	0.05	0.125	895.0
14	5	15.00	0.05	0.125	950.0
15	3	155.00	0.05	0.125	1500.0
16	Base (6)	--	0.01	0.140	5000.0

Notes:

-- – no information available
kcf – kips per cubic foot
ft/sec – feet per second

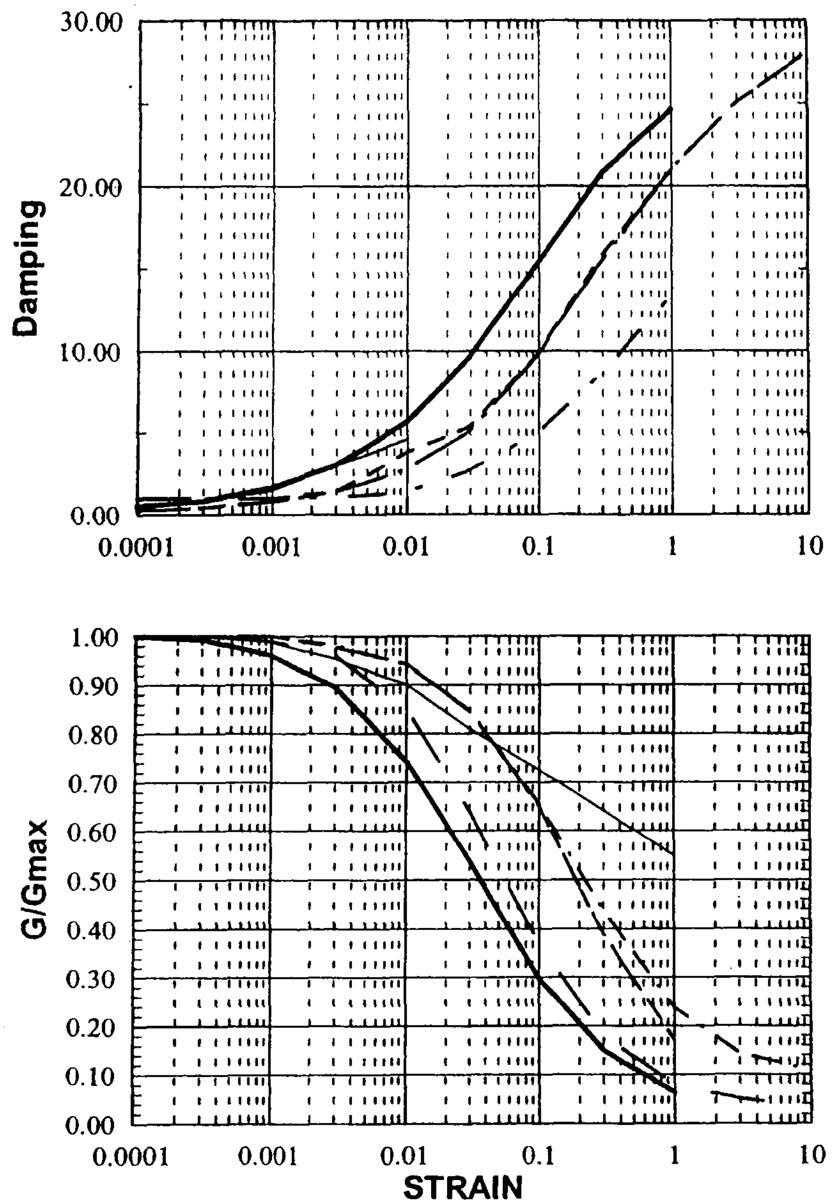
TABLE 4-12c

SHAKE91 ANALYSIS INPUT PARAMETERS FOR SOIL PROFILE 3

Layer No.	Soil Type	Layer Thickness (feet)	Damping	Unit Weight (kcf)	Shear Velocity (ft/sec)
1	1	28.00	0.05	0.130	600.0
2	2	7.00	0.05	0.110	600.0
3	2	18.00	0.05	0.110	600.0
4	3	28.00	0.05	0.130	1050.0
5	4	9.00	0.05	0.125	1050.0
6	4	15.00	0.05	0.125	688.0
7	4	23.00	0.05	0.125	826.0
8	4	20.00	0.05	0.125	902.0
9	4	9.50	0.05	0.125	678.0
10	4	19.50	0.05	0.125	995.0
11	4	22.00	0.05	0.125	800.0
12	4	41.00	0.05	0.125	895.0
13	5	15.00	0.05	0.125	950.0
14	3	155.00	0.05	0.125	1500.0
15	Base (6)	--	0.01	0.140	5000.0

Notes:

-- – no information available
kcf – kips per cubic foot
ft/sec – feet per second



Legend

- Loose Sand (Seed & Idriss 1970)
- Young Bay Mud (Idriss 1990; Pyke, 1995)
- Dense Sand (Modulus: Seed & Idriss 1970, upper range; Damping: Idriss, 1990)
- Old Bay Mud (Idriss, 1990; Pyke, 1995)
- Clay (Modulus: Sun et al., 1988, upper range; Damping: Idriss 1990)
- Rock-Average (Schnabel et al., 1972)

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-14c
MODULUS REDUCTION AND DAMPING RATIO CURVES FOR SOIL PROFILE IN SHAKE91 ANALYSES

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performed for this study to determine soil classifications, unit weights, and shear-wave velocities. Published relations were used to define variations of damping and shear modulus ratio as a function of shear strain for waste materials (Type 1 relations for the upper fill layer, modeling waste material behavior).

The waste in the upper fill layer is generally mixed with granular soils and therefore, the selected properties of the fill materials placed along the disposal area perimeter are considered to be appropriate. Additionally, based on the published data [U.S. Environmental Protection Agency (EPA), 1995], waste material properties are not expected to be too different from the fill properties measured in this investigation. Therefore, the impact of variable amounts of waste on ground motions at the site is negligible due to the similarity of material properties of the mixed soil/waste fill and clean soil fill, and the relatively small thickness of the fill in the disposal area.

Tables 4-12a, 4-12b, and 4-12c summarize the input parameters for different soil layers in the one-dimensional soil profile used in the SHAKE91 analyses. Appendix L provides the input computer files used in SHAKE91 analyses and an example of an output file.

The results of the SHAKE91 site response analyses, providing estimates of the PHGA, are summarized below:

Site Soil Profile	Peak Acceleration of Input Rock Motion (g)	Estimated SHAKE91 Peak Ground Surface Accelerations (g)		
		Set 1 Record	Set 2 Record	Set 3 Record
1 (IR Site 2 Southern Boundary)	0.40	0.41	0.43	0.37
2 (IR Site 2 Western Boundary)	0.40	0.43	0.45	0.34
3 (Area Between IR Sites 1 and 2)	0.40	0.40	0.41	0.43

The above estimated site PHGAs are approximately 4 to 10 percent larger than the preliminary estimate of the site PHGA of 0.40g obtained from the empirical relationship developed by Idriss (1990) for soft soil sites (see Figure 4-12).

The site response analyses provided the maximum site PHGA of 0.45g. This was used in evaluation of the site liquefaction potential and seismically induced settlements. Additionally, the analyses provided average acceleration time histories for different potential sliding mass configurations in seismic deformation analyses of the landfill slopes, which were used in simplified, Newmark-type permanent slope displacement analyses.

4.6.6 Liquefaction Potential Evaluation

Liquefaction is defined as the loss of strength of relatively loose cohesionless (generally sandy), saturated soils when the pore water pressure in the soil reaches the overburden pressure due to strong ground shaking in an earthquake. The primary factors that influence the potential for liquefaction include groundwater table elevation, soil type and grain size characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The zone in which the occurrence of liquefaction may impact performance of a structure is generally considered to be the upper 50 feet below the existing ground surface. Liquefaction potential is greatest in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Clayey (cohesive) soils or soils that possess clay particles ($d<0.005$ mm) in excess of 20 percent or liquid limit of larger than 36 percent (Finn, et al., 1994; Seed and Idriss, 1982) are generally not considered to be susceptible to liquefaction, nor are those soils that are above the static groundwater table.

Based on the field exploration data at IR Site 2 and the Additional Investigation Area [the measured soil penetration resistance (SPT-N values), CPT-tip resistance, and the observed soil types and groundwater depth], the laboratory test results (index properties), and liquefaction potential evaluation analyses, silts and sands underlying the site may be susceptible to liquefaction. Liquefaction susceptibility of these soils depends on their consistency/relative density and earthquake duration and shaking level (for example, earthquake magnitude and cyclic shear stresses developed in the soil). As such, soils in Stratum I (fill) and some of the non-plastic, granular soils interbedded in Stratum II (for example, non-plastic silty soils in Young Bay Mud sediments) are considered potentially liquefiable at this site. Based on the observed SPT blow counts (generally greater than 30), the measured CPT-tip resistance, and the anticipated level of ground motions at the site, the Merritt Sand layer (Stratum III) is generally non-liquefiable.

Standard procedures for evaluating soil liquefaction potential are primarily based on empirical relationships between soil penetration resistance, particularly SPT-N values, cyclic shear stress ratio, and other factors as presented by Seed and others (Seed and Idriss, 1982; Seed, 1986; Seed and Harder, 1990). Similarly, several researchers including Robertson and Campanella (1985), Seed and DeAlba (1986), Shibata and Teparaska (1988), Stark and Olson (1995), Robertson and Wride (1997) have presented methods that follow the same type of procedures, but use CPT penetration resistance values (Q_t) in place of SPT-N values. Most recently, these CPT-based procedures have been summarized in the National Center for Earthquake Engineering Research (NCEER) Workshop on Evaluation of Liquefaction Resistance of Soils (Youd and Idriss, 1997). The CPT has become very popular due to its greater repeatability and the continuous nature of its profile. The above empirical procedures have been calibrated based on documented case histories of liquefaction and non-liquefaction.

4.6.6.1 Liquefaction Analysis Approach

The liquefaction analysis was based on the recent published methods (Youd and Idriss, 1997; Martin and Lew, 1999). The Southern California Earthquake Center (SCEC) 1999 report provides recommended procedures for implementation of CDMG Special Publication 117 (CDMG, 1997). The liquefaction potential of the subject site was analyzed utilizing a maximum peak site acceleration of about 0.45g for a magnitude 7.9 seismic event, or an equivalent weighted peak ground acceleration of approximately 0.50g for a magnitude 7.5 earthquake. The site design PHGA and earthquake magnitude were scaled to equivalent values of approximately 0.50g and 7.5, respectively, using a "Magnitude Weighting Factor" (MWF) developed by Idriss (1998). The analysis was performed using a groundwater elevation of about 2 feet msl, which is considered to be a relatively conservative estimate of a high groundwater table.

The basic evaluation procedure developed by Seed and Idriss (1982), as applied to this study, involves the following three basic steps:

1. Estimating cyclic shear stress induced by earthquake ground motions [or cyclic stress ratio (CSR)] at different depths using a simplified approach. The intensity and duration of ground shaking and the variations of earthquake-induced shear stresses with depth were incorporated in the evaluation.
2. Estimating cyclic shear resistance [or cycling resistance ratio (CRR)] at different depths (namely, cyclic shear stresses that would be required to cause liquefaction in a number of uniform shear stress cycles corresponding to the design earthquake ground motions). This was accomplished using available empirical correlations between documented cases of field performance (liquefaction versus non-liquefaction) and normalized soil penetration resistance, properly corrected for confining pressure, soil grain characteristics, fines fraction, and in situ testing procedures. The soil type, in situ conditions, seismic and geologic histories of the deposit, and the initial effective stress conditions are approximately incorporated in the evaluation. Empirical relations and charts to estimate CRR as a function of corrected SPT-N values or CPT tip resistance are presented in recent publications by Youd and Idriss (1997) Martin and Lew (1999).
3. Comparing shear stresses induced by the earthquake (Step 1) with those required to cause liquefaction (Step 2), (or CSR versus CRR) to evaluate the potential zone (or depth range) of liquefaction in the soil deposit, corresponding to places where induced cyclic shear stresses exceed those required to cause liquefaction (CSR greater than CRR). A factor of safety against liquefaction may be defined as the ratio of CRR to CSR. Therefore, if the factor of safety for a soil layer is less than 1, then the soil layer is potentially liquefiable.

4.6.6.2 Data Evaluation

Soil penetration resistance data were modified for use in the liquefaction potential analysis. SPT field blow counts were modified to include a correction for normalizing to an overburden pressure of 2,000 pounds per square foot (psf), or 1 tons per square foot (tsf), which approximately corresponds to 100 kilopascals, correction for the amount of fines (percent of soil passing No. 200 sieve), hammer energy, hammer type, sampler type, and lining (or no lining) of SPT samplers.

CPT-tip resistance and friction ratio were corrected and normalized as proposed by Robertson and Wride (1997). An example of applying the integrated CPT profiles for estimating the CRR at one location at the IR Site 2 landfill site is illustrated in Figure 4-15 for CPT Location C-2-6. This CPT profile summarizes measured cone-tip resistance (Q_t) and friction ratio, as well as interpreted soil behavior type index (I_c), and "clean-sand equivalent" normalized soil penetration resistance [$(q_{c1N})_{cs}$]. CPT profiles for CPT Locations C-2-1 through C-2-15 in IR Site 2 and CPT Locations C-2-16 through C-2-21 in the area between IR Sites 1 and 2 are presented in Appendix L.

The integrated CPT profiles presented in Appendix L were used to evaluate site soils liquefaction potential and to estimate liquefaction-induced deformations (settlement and lateral spread). These profiles provide data on soil consistency (density and/or stiffness), other properties such as soil types and fines content, and sandy soils CRR for equivalent magnitude 7.5 earthquake ($CRR_{7.5}$). The $CRR_{7.5}$ is compared to the CSR induced by the design earthquake for subsurface saturated sandy soils corrected for magnitude using the MWF ($CRR_{7.5}$). If the computed value of CSR is greater than the CRR of a saturated sandy soil layer (FS = [$CRR_{7.5}/CSR_{7.5}$] < 1), potential of earthquake-induced liquefaction may exist. These sandy soil layers are typically characterized in the CPT sounding records by a normalized friction ratio less than approximately 2, and by soil index less than 2.6.

In situ testing at the site also included SPT measurements. Raw (uncorrected) soil penetration resistance (SPT-N) values obtained from the test borings are presented in the test boring logs in Appendix C. The test borings were drilled using mud rotary drilling technique, which provide generally more reliable SPT-N values. Samples were taken at approximately 5-foot center-to-center intervals and at major changes in strata. Logging of the borings was based on the general description of the soil encountered in the spoon and the interbedding of the subgrade may not have been reflected in the boring logs. In some cases, the sampler was driven in proximity of overlying or underlying cohesive layers. As a result, some of the N values are likely to be influenced by the softer cohesive soils and not fully reflect the consistency of the cohesionless soil layers. Soil descriptions and SPT-N values obtained from borings drilled adjacent to or close to the CPT soundings were added to the cross sections developed using the CPT sounding profiles for comparison (see Figures 4-8a through 4-8i).

4.6.7 Liquefaction-Induced Deformations

The effect of earthquake-induced liquefaction in a saturated sandy soil in general, may vary widely depending on the CSR and CRR of the soil layer and could range from very limited ground deformation for high CRR values (for example, greater than 0.50; with minimum impact

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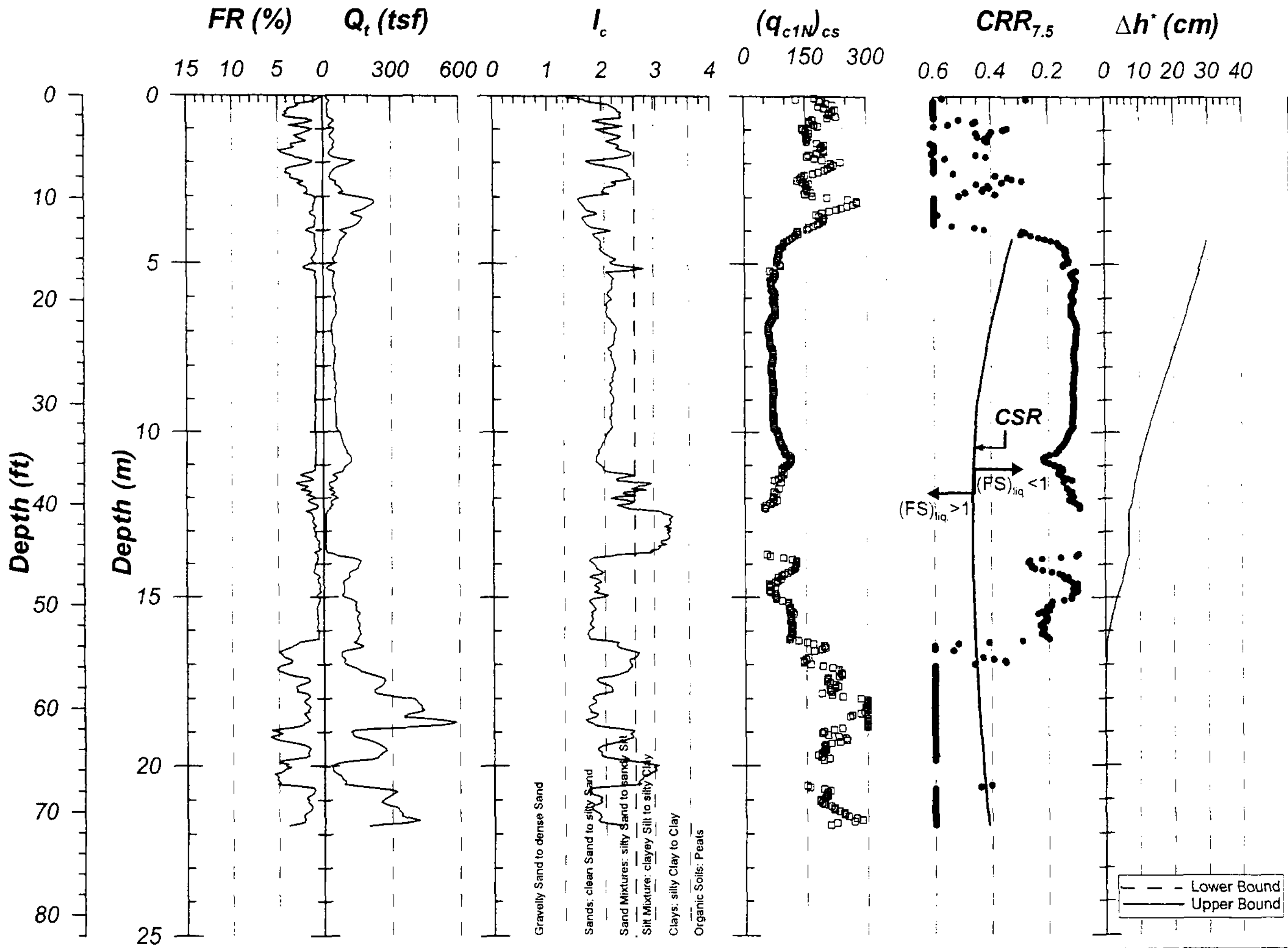

NOTE:
$(q_{c1N})_{cs}$ and CRR plots are truncated at 300 and 0.6, respectively.
* Δh is liquefaction-induced settlement and does not include earthquake-induced settlement of unsaturated soils.

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-15
INTEGRATED CPT METHOD FOR ESTIMATING SUBSURFACE STRATIFICATION AND SETTLEMENT AT C-2-5

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to overlying landfill structures), to flow-type failure for very low CRR values (for example, less than 0.1).

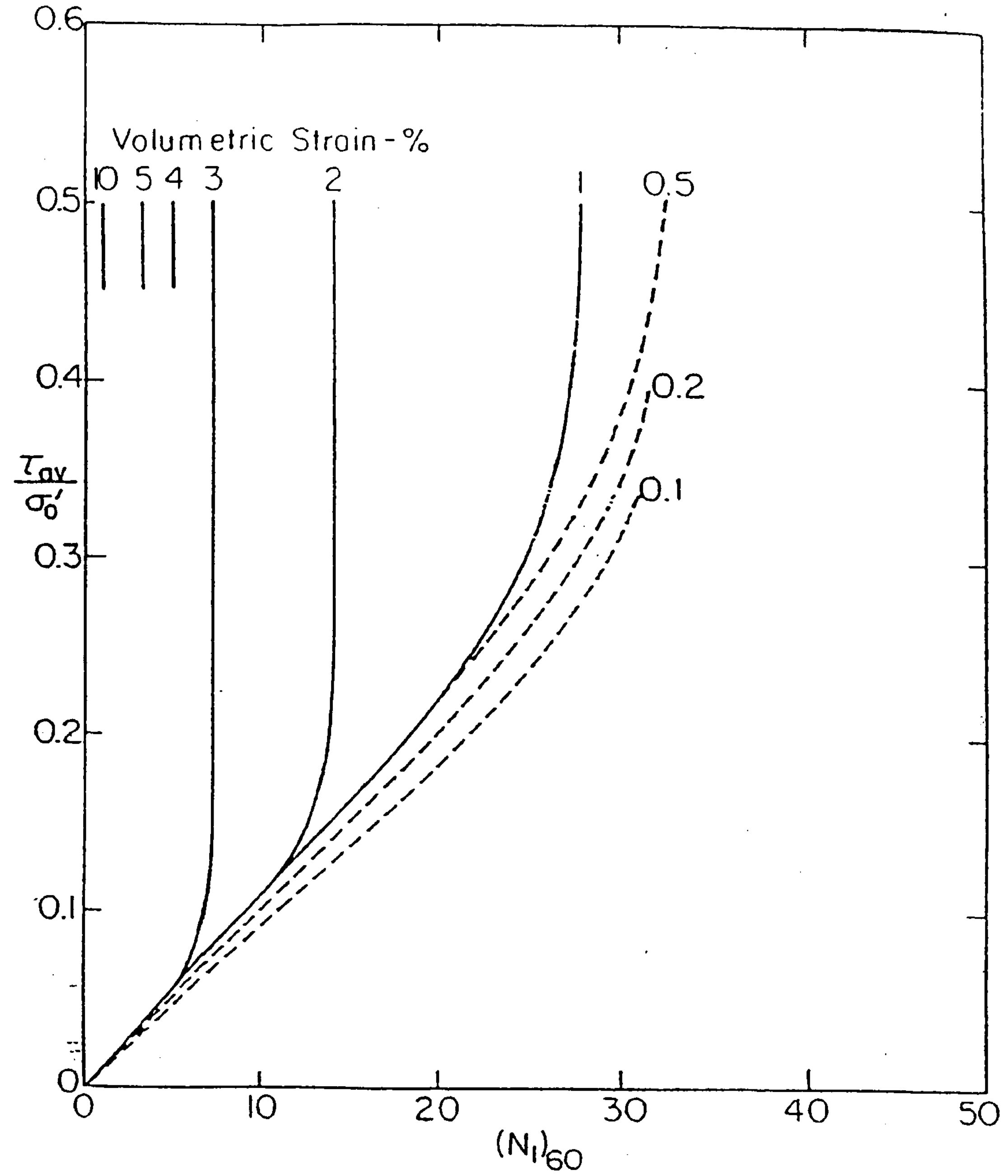
Consequently, emphasis in this study was placed in assessing the likely order of magnitude of these deformations, namely, settlement and horizontal deformations at ground surface. These evaluations consisted of simplified, empirical, order-of-magnitude-type of estimates based on available soil penetration resistance data and known empirical correlations to aid in assessing whether remedial measure would, or would not, be necessary.

Empirical procedures were used to provide rough estimates of liquefaction-induced ground settlements and lateral displacements. These methods use correlations between soil penetration data (CPT-q_c and SPT-N values) with results of well-documented laboratory tests and site design ground motion parameters to estimate volumetric strains in soils and accumulated ground settlements. Lateral spread displacements were also estimated from empirical relations developed based on soil penetration data, earthquake ground motion parameters, and a dataset of well-documented case histories of field performance where earthquake motions and ground deformations were measured during and after an earthquake.

Liquefaction-induced ground deformations (settlements and lateral spread) are estimated to occur primarily within the upper fill soils (generalized Stratum I), and possibly in localized sandy soil layers or lenses within the underlying soils. The thickness of those liquefaction-vulnerable sediments contributing to ground deformations at IR Site 2 and the area between IR Sites 1 and 2 is on the order of 10 to 40 feet depending on location. The following subsections summarize the methods used and the estimates of the liquefaction induced settlements and permanent lateral displacements.

4.6.7.1 Liquefaction-Induced Settlements

Estimates of liquefaction-induced permanent vertical strains were made at various CPT sounding and boring locations. Values were calculated where CSR exceeded estimated CRR values (FS < 1). These estimates were made based on available empirical correlations between volumetric strains with corrected soil penetration resistance [equivalent clean-sand CPT $(q_c)_1$ or SPT-N_1 values]. The liquefaction-induced settlements for IR Site 2 were estimated based on the CPT data, because a large amount of CPT data, providing a relatively continuous characterization of the site soils, were collected at the site. The samples collected from the boreholes drilled at the site were also used to obtain additional data (fines content, plasticity index, and so forth) for use in liquefaction analyses. The CPT-based settlements were estimated using the relation developed by Ishihara and Yoshimine (1992) based on the laboratory cyclic simple shear tests performed on sands (Figure 4-16). The estimated values of the volumetric strains were multiplied by the approximate thickness of potentially liquefiable soil layers to calculate permanent ground surface settlements, as summarized in Appendix L, Figures L1 through L24. Figure 4-17 illustrates the correlation developed by Tokimatsu and Seed (1987) for



after Tokimatsu and Seed, 1987

Figure 4-16
CYCLIC STRESS RATIO VERSUS VOLUMETRIC STRAIN FOR SATURATED CLEAN SANDS AND M=7.5

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SOURCE: HUSHMAND ASSOCIATES, INC.

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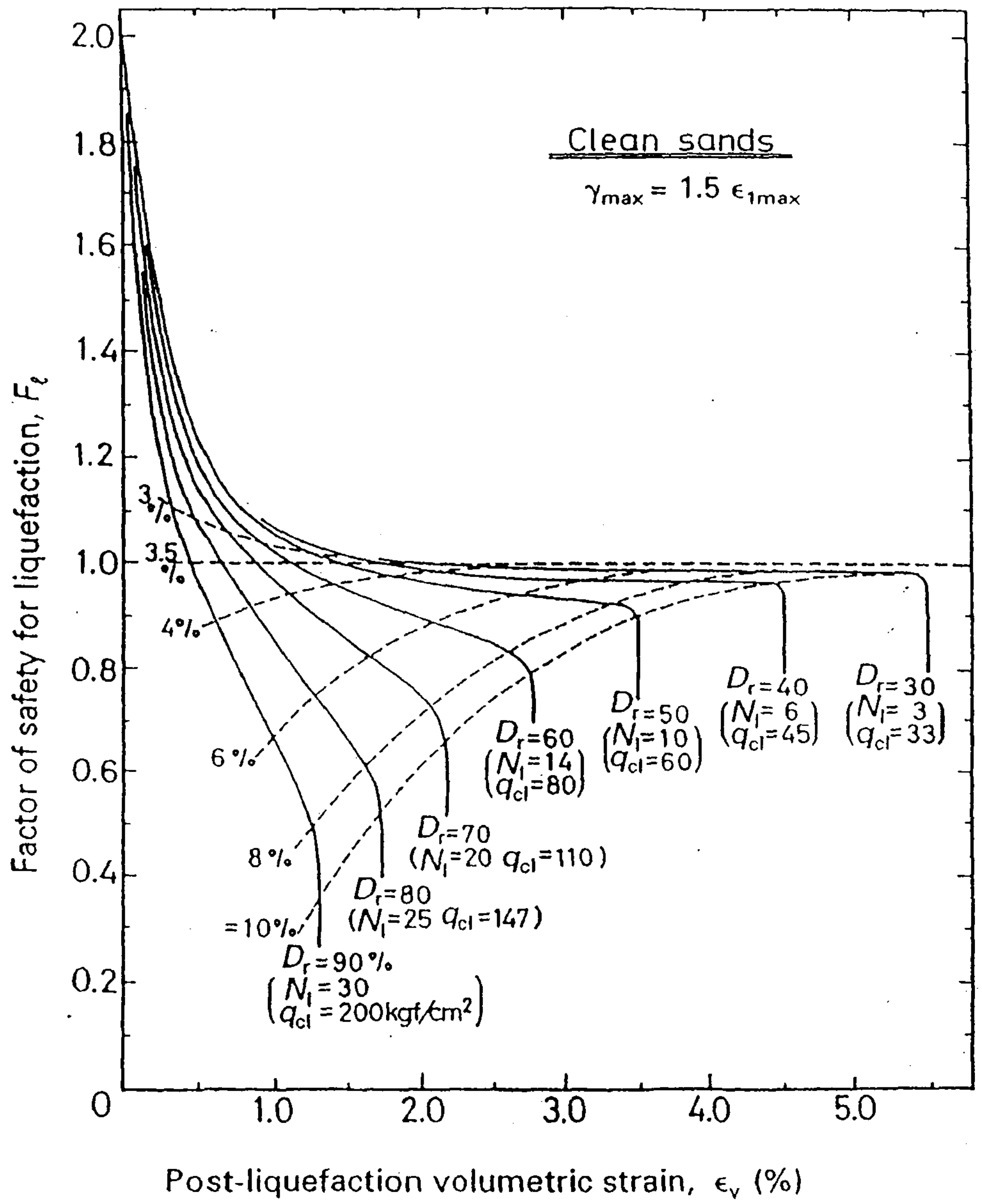

after Ishihara and Yoshimine (1992)

Figure 4-17
POST-LIQUEFACTION VOLUMETRIC STRAIN AS A FUNCTION OF FACTOR OF SAFETY

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estimating liquefaction-induced volumetric strains based on soil penetration resistance SPT-N_1 values. CPT-tip resistance data may also be used with Figure 4-17 to estimate settlements. This is done by converting the CPT-tip resistance to an equivalent SPT value using available CPT-SPT soil penetration resistance correlations (Seed and Idriss, 1982). Table 4-13 summarizes the estimated liquefaction-induced settlements at the upland CPT locations. The maximum estimated liquefaction-induced settlement is on the order of 12 inches. An additional settlement of approximately 4 to 6 inches could occur due to possible liquefaction/consolidation of silty soils in Young Bay Mud sediments. Therefore, the total seismic/liquefaction-induced settlement is estimated a maximum of 18 inches.

Estimates of undrained residual shear strength of liquefied sandy soil layers were developed based on empirical correlations between this strength parameter and corrected soil penetration resistance (SPT-$N_{1(60)}$), developed by Seed and Harder (1990). Based on the estimated residual strength of Young Bay Mud and liquefaction-susceptible soils (see Table 4-6a), evaluations were made of post-earthquake static stability conditions. These results are presented in Section 4.6.8 of this report.

Seismically induced settlements of non-liquefiable soils (deeper clayey soils and Merritt Sand/Bay Sediments) were estimated to be negligible because of the cohesive nature of clayey soils and relatively high SPT blow counts recorded from the Merritt Sand/Bay Sediments.

4.6.7.2 Liquefaction-Induced Permanent Lateral Displacements

Permanent horizontal ground displacements resulting from liquefaction-induced lateral spread were estimated based on an empirical model developed by Bartlett and Youd (1992) (revised by Youd et al., 2002). The model was developed from multiple linear regression analyses of U.S. and Japanese case histories of lateral spread. The magnitude of lateral displacements associated with the presence of a "free face," the condition existent along the western and southern perimeter slopes of IR Site 2 and the Additional Investigation Area, is strongly correlated to height of and distance from the slope free face. Other factors, such as earthquake magnitude, distance to the seismic energy source, thickness of liquefiable sediments, and the fines content and particle size of those sediments are also correlated with ground displacements. Because case history data for displacements greater than approximately 20 feet (6 meters) is not sufficient (observed during only 1964 Niigata, Japan, earthquake with lateral spread of banks of the Shinano River toward the river channel), predicted displacements greater than 20 feet, using the above empirical method, are not reliable. Such large predicted displacements do indicate, however, that displacements are likely to be large.

Thickness of liquefiable sediments (T_{15}) was based on the integrated CPT-based liquefaction evaluation approach developed by Robertson and Wride (1997). The fines content and particle-size (D_{50}) data for liquefiable soils were derived from the results of the laboratory tests (grain-size distribution analyses) performed on soil samples obtained from boreholes drilled at the site

TABLE 4-13

ESTIMATED LIQUEFACTION-INDUCED SETTLEMENTS AT LOCATIONS OF CPTs

CPT Locations	C-2-1	C-2-2	C-2-3	C-2-4	C-2-5	C-2-6	C-2-7
Settlement (inch)	7.6	10.4	10.4	8.4	11.7	11.3	9.3

CPT Locations	C-2-8	C-2-9	C-2-10	C-2-11E	C-2-12A	C-2-13	C-2-14
Settlement (inch)	12.3	8.1	5.7	4.0	3.9	6.4	1.5

CPT Locations	C-2-15A	C-2-16	C-2-17	C-2-18	C-2-19	C-2-20	C-2-21
Settlement (inch)	8.9	8.1	8.4	9.0	8.4	7.0	6.8

Notes:

CPT – cone penetration test

(Appendix H). The design earthquake parameters used in this method are the earthquake magnitude and the site horizontal distance from the seismic energy source. An earthquake magnitude of 7.9 and distance of 19 kilometers to seismic source was used, which correspond to those of the MCE event on the San Andreas Fault as discussed in Section 4.6.5.

Lateral spread displacements of the water front slopes, preliminarily estimated using the above empirical method, appear to greatly exceed 20 feet. These estimated relatively large liquefaction-induced lateral displacements are beyond the limit of the accuracy of the above empirical method (Youd et al., 2002). The potential and magnitude of these deformations are exacerbated by the presence of the relatively steep slope ("wall face") at the water front. In upland areas, say more than approximately 100 to 200 feet from the water front, those deformations are significantly reduced. Lateral displacement calculations are included in Appendix L.

The above estimates are based on preliminary calculations and can be refined/confirmed for detailed design evaluations of the site remedial measures using numerical modeling methods.

4.6.8 Seismic Slope Stability

The state-of-practice in seismic stability evaluation of landfill slopes generally includes computation of seismically induced permanent slope displacements using simplified (Newmark-type) methods of analysis (Newmark, 1965; Franklin and Chang, 1977; Makdisi and Seed, 1978; Hynes-Griffin and Franklin, 1984; Bray et al., 1998).

The analyses were conducted in the following evaluation/computational sequence:

- Assessment of analysis soil profile and parameters
- Selection of analysis sections
- Static slope stability and selection of critical failure surfaces
- Pseudo-static slope stability and evaluation of yield acceleration coefficient
- Estimation of average acceleration time history of potential slide mass using the one-dimensional site response analysis program, SHAKE91
- Estimation of seismically induced permanent deformations for the MCE design earthquake event

These six stages are described below.

Analysis Soil Profile and Parameters

The analysis soil profile consists of the proposed landfill cover, existing near-surface fill and various foundation soil strata as shown in the cross sections shown in Figures 4-8a through 4-8i. Locations of these cross sections are presented in plan view in Figure 4-7. Stratigraphic conditions were discussed in Section 4.4.1.

The details regarding the assumed cover are included in Appendix M. These include soil type, geometry, and material properties. The assumptions regarding the soil cover will impact the stability analyses. However, the impact is expected to be minor. More detailed analyses may have to be performed after the final cover design to verify the stability results.

Based on the findings of the field investigation, laboratory test results, and review of published data, geotechnical parameters were developed for analysis purposes and are included in Table 4-6a. The data from previous studies at the site (TtEMI, 1999; 2001) or investigations in the vicinity of the site (Fugro-EMI, 2001a; 2001b), including published data on properties of Young Bay Mud (Pyke, 1989), were also used in the derivation of the site geotechnical parameters. Consolidated-drained (CD) shear strength properties of sands and clays in the upper four soil strata at the site were derived from results of laboratory tests (Appendix H). Direct shear tests and consolidated-undrained (CU) triaxial shear tests with pore pressure measurement were performed to estimate CD shear strength properties used in long-term static stability analyses.

CU shear strength parameters of Young Bay Mud (soft to very soft clays) and Old Bay Mud (stiff to very stiff clays) were estimated based on the results of field and laboratory tests performed for this project and a review of published data (Fugro-EMI, 2001a; 2001b; Pyke, 1989). The Stress History and Normalized Soil Engineering Properties (SHANSEP) method (Ladd and Foott, 1974; Ladd, 1991), laboratory and published data, and correlations with Liquidity Index and Plasticity Index provide an estimated value of S_U / σ'_{v0} ratio in the range of 0.2 to 0.3 for Young Bay Mud and approximately 0.3 for Old Bay Mud. The conservative values of 0.2 and 0.3 were used for Young Bay Mud and Old Bay Mud clays, respectively in Table 4-6a, and to calculate shear strength properties for normally consolidated clays.

The post-earthquake (residual) undrained shear strength properties of liquefiable granular soils in the upper fill layer and Young Bay Mud underlying the fill layer were estimated from results of field and laboratory tests for this project and a literature search for properties of Young Bay Mud. The post-earthquake strength properties were used in post-earthquake and seismic (pseudo-dynamic) slope stability analyses.

The residual undrained shear strength (S_u) of liquefied granular soils of the upper fill layer was estimated from the empirical approach developed based on correlations between SPT blow counts and apparent residual strength back-calculated from observed flow slides (Seed and Harder, 1990). This empirical relationship is commonly used in practice (Martin and Lew, 1999). Mean or lower-bound values of the data range used to develop plots of the residual undrained shear strength versus equivalent clean sand SPT blow count were used in estimating strength properties.

The post-earthquake residual undrained shear strength of Young Bay Mud was estimated from published data (Ramanujam et al., 1978; Pyke, 1989), particularly results of extensive field and laboratory tests performed on Young Bay Mud as part of the geotechnical investigation performed recently for the SFOBB project located less than 2 miles from Alameda Point's IR Site 1 (Fugro-EMI, 2001b).

Cyclic triaxial and simple shear tests performed on samples of clayey soils used in design and construction of an earth dam and on Young Bay Mud samples as part of the extensive geotechnical investigation for the SFOBB project indicated an approximate 20 percent reduction in undrained shear strength of clay samples following the cyclic loading (Ramanujam et al., 1978; Fugro-EMI, 2001b). The samples were loaded to confining stresses approximately equal to the in situ field stresses and sheared using a cyclic load. The cyclic shear stress amplitudes and number of cycles of loading in the tests were representative of the intensity and duration of the shaking produced by the design earthquake. Additionally, results of miniature vane shear tests performed on Young Bay Mud samples (Appendix H) are in agreement with residual undrained shear strength properties used in post-earthquake stability analyses.

Engineering properties of cover materials are unknown at this time. However, the following shear strength properties can be used in the stability analysis: friction angle; $\phi = 34°$; cohesion, C = 200 psf. These values are typically used for compacted cover material composed of medium dense silty to clayey sand (SM-SC). A table of typical properties of compacted soils is included in Appendix M.

Analysis Sections

Various IR Site 2 cross sections (perpendicular to the western and southern shorelines) were analyzed for slope stability.

Cross Sections C-C' through I-I' (see Figure M-1 and cross sections in Appendix M) represent critical slope configurations and landfill geometry. The cross sections are shown in Figures 4-8c through 4-8i, and their locations in plan view are shown in Figure 4-7. Cross sections E-E', D-D', C-C', and I-I' are oriented approximately west-east, perpendicular to the western shoreline. Cross sections H-H', G-G' and F-F' are oriented approximately north-south perpendicular to the southern shoreline.

The analysis included an evaluation of existing conditions as well as the effect of the proposed cover system. A 4-foot-thick soil cover was modeled for the cover system. Cross Section I-I' is not modeled with a cover system because it is located on a former air strip, and no additional cover is planned.

Static Stability

Conventional two-dimensional limit-equilibrium stability analyses were performed for all sections shown in Appendix M. The computer program, PC-STABL-5M (Achilleos, 1988), was used to calculate the factors of safety against potential failure. The program uses two-dimensional limiting equilibrium theory to provide general solutions to slope stability problems. Both circular and noncircular potential sliding surfaces can be pre-specified or randomly generated. Modified Janbu and Bishop methods of analysis were used for this study (Achilleos, 1988). Most critical surfaces identified during an initial extensive search based on the simplified Janbu method of analysis were subsequently analyzed using the more rigorous Spencer's method of analysis. The Modified Bishop and Janbu methods are considered less rigorous methods because they do not satisfy both force and moment equilibrium simultaneously. These methods are generally conservative compared with the more rigorous Spencer's method (Achilleos, 1998), and they typically result in lower factors of safety than the more rigorous methods (Duncan, 1992).

Appendix M presents plots illustrating geometries of the IR Site 2 perimeter slope cross sections and the ten most critical potential failure planes searched by the program, as well as computed factors of safety. The failure plane with the lowest factor of safety is identified with two arrows at its initiation and termination points.

The most critical potential failure mechanism considered was either a circular failure or a wedge (block) failure plane starting at the landfill surface, passing through the proposed landfill cover and the existing underlying fill, and then sliding mostly within the Young Bay Mud away from the shoreline toward San Francisco Bay. Figure 4-18 illustrates a typical slope stability analysis model showing the relative location of the most critical potential failure plane with respect to the previously discussed geologic units. Three different loading cases were analyzed for all sections. These cases included: 1) static (pre-earthquake) stability analysis, 2) pseudo-static stability analysis to compute yield accelerations (the pseudo-static earthquake acceleration resulting in a factor of safety of approximately 1.0), and 3) the post-earthquake static stability analysis. The first case was analyzed using the initial pre-earthquake strength undrained properties of the soil materials (see Table 4-6a). Long-term static stability analyses using CD shear strength properties (c' and ϕ') were performed for the critical Cross Section C-C'. Long-term stability analyses simulated conditions where the materials had enough time to dissipate excess pore water pressure. These analyses resulted in higher factors of safety compared to analyses performed using CU shear strength properties as shown in Table 4-14. The second case was analyzed using the average between the long term (pre-earthquake) and the post-earthquake strength properties. The post-earthquake case was analyzed using the residual shear strength properties of the Young Bay Mud and the liquefied granular soils (reduced strength properties due to strong ground shaking, see Table 4-6a).

Results of slope stability evaluations are included in Appendix M and are summarized in Table 4-14. As discussed in Section 1.4 of the report, for all static (long-term) stability conditions, the minimum acceptable factor of safety is 1.5 (Title 27 CCR). This criterion was satisfied for all cross sections, except Cross Section C-C'. The minimum pre-earthquake static factor of safety at Section C-C' is approximately 1.46. Although all landfill slopes have factors

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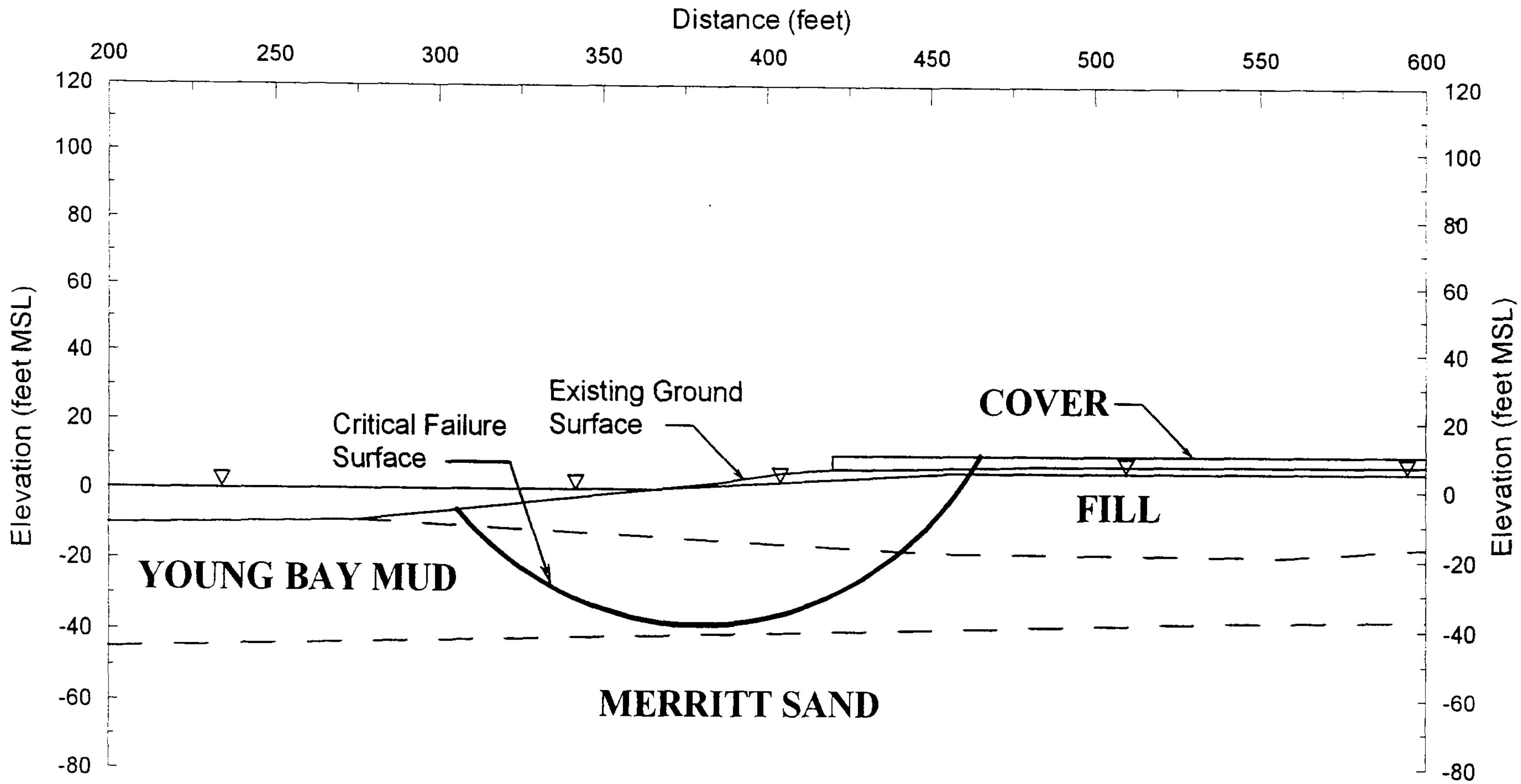

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MSL Mean Sea Level

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-18
TYPICAL SLOPE STABILITY ANALYSIS MODEL SHOWING A POTENTIAL FAILURE PLANE

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TABLE 4-14

SUMMARY OF SLOPE STABILITY ANALYSIS RESULTS

Analysis Cross Section	Static and Pseudo-Static Factor of Safety	Remarks	Yield Acceleration K_y (g), and Seismic Permanent Displacement * (feet)	
			K_y (1)	* (3)
[C-C']	1.46 [S](2)	Static Slope Stability, Circular Failure Surface	--	--
	1.55 [S](2)	Static Slope Stability, Block Failure Surface	--	--
	3.18 [S](2)	Long-term Slope Stability, Circular Failure Surface	--	--
	3.74 [S](2)	Long-term Slope Stability, Block Failure Surface	--	--
	1.00 [S](2)	Pseudo-static, Circular Failure Surface, Averaged Existing and Post-earthquake, Soil Properties	0.03	19
	1.00 [S](2)	Pseudo-static, Block Failure Surface, Averaged Existing and Post-earthquake, Soil Properties	0.03	19
	1.06 [S](2)	Post-earthquake, Static Slope Stability, Circular Failure Surface	--	--
	1.18 [S](2)	Post-earthquake, Static Slope Stability, Block Failure Surface	--	--
[D-D']	1.69 [S](2)	Static Slope Stability, Circular Failure Surface	--	--
	1.79 [S](2)	Static Slope Stability, Block Failure Surface	--	--
	1.00 [S](2)	Pseudo-static, Circular Failure Surface, Averaged Existing and Post-earthquake, Soil Properties	0.03	19
	1.00 [S](2)	Pseudo-static, Block Failure Surface, Averaged Existing and Post-earthquake, Soil Properties	0.04	14
	1.13 [S](2)	Post-earthquake, Static Slope Stability, Circular Failure Surface	--	--
	1.19 [S](2)	Post-earthquake, Static Slope Stability, Block Failure Surface	--	--
[E-E']	1.77 [S](2)	Static Slope Stability, Circular Failure Surface	--	--
	2.07 [S](2)	Static Slope Stability, Block Failure Surface	--	--
	1.00 [S](2)	Pseudo-static, Circular Failure Surface, Averaged Existing and Post-earthquake, Soil Properties	0.04	14
	1.00 [S](2)	Pseudo-static, Block Failure Surface, Averaged Existing and Post-earthquake, Soil Properties	0.04	14
	1.02 [S](2)	Post-earthquake, Static Slope Stability, Circular Failure Surface	--	--
	1.12 [S](2)	Post-earthquake, Static Slope Stability, Block Failure Surface	--	--
[F-F']	2.11 [S](2)	Static Slope Stability, Circular Failure Surface	--	--
	2.58 [S](2)	Static Slope Stability, Block Failure Surface	--	--
	1.93 [S](2)	Long-term Slope Stability, Circular Failure Surface	--	--
	2.71 [S](2)	Long-term Slope Stability, Block Failure Surface	--	--
	1.00 [S](2)	Pseudo-static, Circular Failure Surface, Averaged Existing and Post-earthquake, Soil Properties	0.06	10
	1.00 [S](2)	Pseudo-static, Block Failure Surface, Averaged Existing and Post-earthquake, Soil Properties	0.06	10
	0.86 [S](2)	Post-earthquake, Static Slope Stability, Circular Failure Surface	--	--
	1.30 [S](2)	Post-earthquake, Static Slope Stability, Block Failure Surface	--	--

TABLE 4-14

SUMMARY OF SLOPE STABILITY ANALYSIS RESULTS

Analysis Cross Section	Static and Pseudo-Static Factor of Safety	Remarks	Yield Acceleration K_y (g), and Seismic Permanent Displacement * (feet)	
			K_y (1)	* (3)
[G-G']	1.72 [S](2)	Static Slope Stability, Circular Failure Surface	--	--
	2.48 [S](2)	Static Slope Stability, Block Failure Surface	--	--
	1.00 [S](2)	Pseudo-static, Circular Failure Surface, Averaged Existing and Post-earthquake, Soil Properties	0.06	10
	1.00 [S](2)	Pseudo-static, Block Failure Surface, Averaged Existing and Post-earthquake, Soil Properties	0.07	8
	1.08 [S](2)	Post-earthquake, Static Slope Stability, Circular Failure Surface		
	1.38 [S](2)	Post-earthquake, Static Slope Stability, Block Failure Surface	--	--
[H-H']	2.14 [S](2)	Static Slope Stability, Circular Failure Surface	--	--
	2.36 [S](2)	Static Slope Stability, Block Failure Surface	--	--
	1.00 [S](2)	Pseudo-static, Circular Failure Surface, Averaged Existing and Post-earthquake, Soil Properties	0.11	4
	1.00 [S](2)	Pseudo-static, Block Failure Surface, Averaged Existing and Post-earthquake, Soil Properties	0.09	5
	1.43 [S](2)	Post-earthquake, Static Slope Stability, Circular Failure Surface	--	--
	1.93 [S](2)	Post-earthquake, Static Slope Stability, Block Failure Surface	--	--
[I-I']	2.08 [S](2)	Static Slope Stability, Circular Failure Surface	--	--
	2.30 [S](2)	Static Slope Stability, Block Failure Surface	--	--
	1.00 [S](2)	Pseudo-static, Circular Failure Surface, Averaged Existing and Post-earthquake, Soil Properties	0.06	9
	1.00 [S](2)	Pseudo-static, Block Failure Surface, Averaged Existing and Post-earthquake, Soil Properties	0.06	9
	1.63 [S](2)	Post-earthquake, Static Slope Stability, Circular Failure Surface	--	--
	1.94 [S](2)	Post-earthquake, Static Slope Stability, Block Failure Surface	--	--

Notes:

(1) K_y Yield acceleration, defined as the value of the horizontal acceleration resulting in a pseudo-static factor of safety equal to unity.
(2) [S] Spencer's "rigorous" method of analysis, used for most critical cases and loading conditions.
(3) * Seismically induced permanent displacement computed based on the procedure using the Newmark's double-integration method of analysis (Newmark, 1965).

of safety greater than 1.0 and are statically stable, remediation measures involving geotechnical improvements of existing site conditions are needed to increase the static factors of safety to at least 1.5.

For post-earthquake stability conditions, according to the United States Army Corps of Engineers (USACE) Manual EM 1110-2-1913 (USACE, 2000a), the minimum acceptable factor of safety is 1.0. This criterion was satisfied for all cross sections except Cross Section F-F' where the fill is thicker. Minor remediation is required to improve stability.

Potential Sliding Mass and Yield Accelerations

Yield accelerations were subsequently computed from a series of pseudo-static analyses. As with the static cases, the pseudo-static slope stability analyses showed that the most critical potential failure mechanism considered is a circular failure or a wedge (block) failure plane sliding mostly through the Young Bay Mud and then through the existing overlying fill and the proposed landfill cover.

The computed yield acceleration coefficient, K_y, represents a limiting value of the horizontal seismic coefficient beyond which movement would likely occur (the seismic coefficient resulting in a factor of safety equal to 1.0).

Table 4-14 summarizes the computed yield acceleration coefficients obtained from pseudo-static stability analyses. Plots of the potential failure planes and values of computed yield accelerations are also provided in Appendix M. The results of these analyses show that the minimum yield acceleration coefficient is approximately 0.03 and occurs at Cross Sections C-C' and D-D'. Relatively low values of yield acceleration (0.04 to 0.11) occur at other analysis cross sections (see Table 4-14).

Dynamic Site Response

As discussed earlier, dynamic response of the landfill and average acceleration time history of the potential sliding mass was evaluated for three representative input ground motions (Section 4.6.3). Although the site has a two-dimensional geometry along the shoreline, the state-of-practice in most cases is, to compute one-dimensional dynamic response of a representative soil/landfill waste column, which generally provides a conservative estimate of the site seismic response.

Using the computed time histories of shear stresses and accelerations for different soil layers within the soil/landfill waste column, average accelerations of the potential slide mass were computed.

Seismically Induced Permanent Displacement Analyses

The effects of earthquake shaking on the landfill slopes were evaluated by estimating seismically induced permanent displacements using Newmark-type, pseudo-dynamic, double-integration deformation analysis methods. During an earthquake, over numerous cycles of loading, a slide mass can move incrementally along a potential failure plane through displacement accumulation. The maximum seismic-induced displacement depends primarily on the characteristics of the site design earthquake ground motion (peak acceleration, frequency content, and duration) and the dynamic response characteristics of the landfill and its foundation soils.

Figures 4-19a, b, and c summarize the results of the estimated seismically induced permanent displacements (computed using a Newmark-type double-integration method applied to the average acceleration time history of the potential sliding mass) versus the yield seismic coefficient (K_y). These analyses use, as input, the average acceleration time history of the potential sliding mass estimated from the one-dimension dynamic SHAKE91 response analyses. Figures 4-19a, b, and c provide computed seismically induced permanent slope displacements versus K_y, for the average acceleration time histories computed from SHAKE91 site response analyses for Soil Profiles 1 through 3 and the three selected input rock motions (Section 4.6.5.1). Table 4-14 summarizes the estimated K_y values (as a fraction of the gravitational acceleration, g) and potential slide mass displacements in feet. Computed K_y values were approximately 0.03 to 0.11. For these yield acceleration values, the calculated seismically induced displacements during the MCE design event are on the order of 4 to 19 feet based on a maximum peak ground surface site acceleration of 0.45g for an earthquake magnitude of 7.9.

Newmark's double-integration analysis method was also used to estimate the seismically induced lateral slope deformations at Alameda Point due to the ground shaking during the 1989 magnitude 7.1 Loma Prieta Earthquake. The ground surface earthquake acceleration time history recorded at Alameda Point's Hangar 23 seismic station [Pacific Engineering and Analysis (PEA), 1997] was used to estimate seismic displacements at the site (Figure 4-19d). The estimated seismic deformations were in the range of 13 to 19 inches corresponding to K_y values of 0.03 to 0.01. These displacements are computed using the ground surface acceleration time history, which generally results in larger estimates of seismically induced slope displacements compared to the average acceleration time history of a potential failure mass. The estimated slope displacements are consistent with the deformations observed after the earthquake at the site (Section 4.6.3).

4.6.9 Summary of Seismic Hazards

Seismic hazards at IR Site 2 and the Additional Investigation Area include liquefaction potential and seismic slope instability. Artificial fill material, placed at the site from dredging operations, which extends to a depth of approximately 40 feet bgs, was determined to be susceptible to earthquake-induced liquefacation. Liquefaction-induced settlements were estimated to be around

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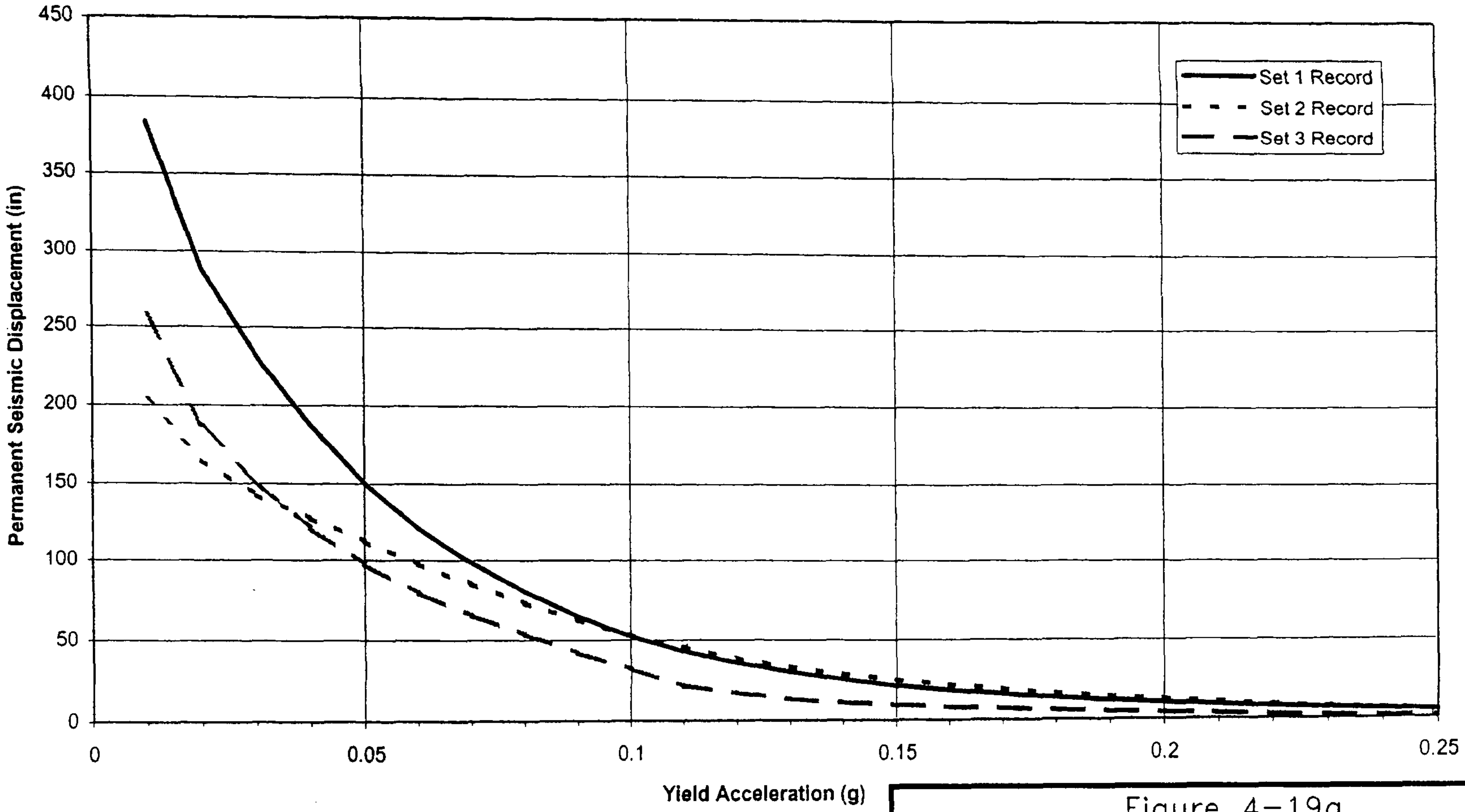

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-19a
SEISMICALLY INDUCED SLOPE DISPLACEMENTS VERSUS YIELD ACCELERATION (SOUTH SIDE OF IR SITE 2)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER ENVIRONMENTAL CORPORATION

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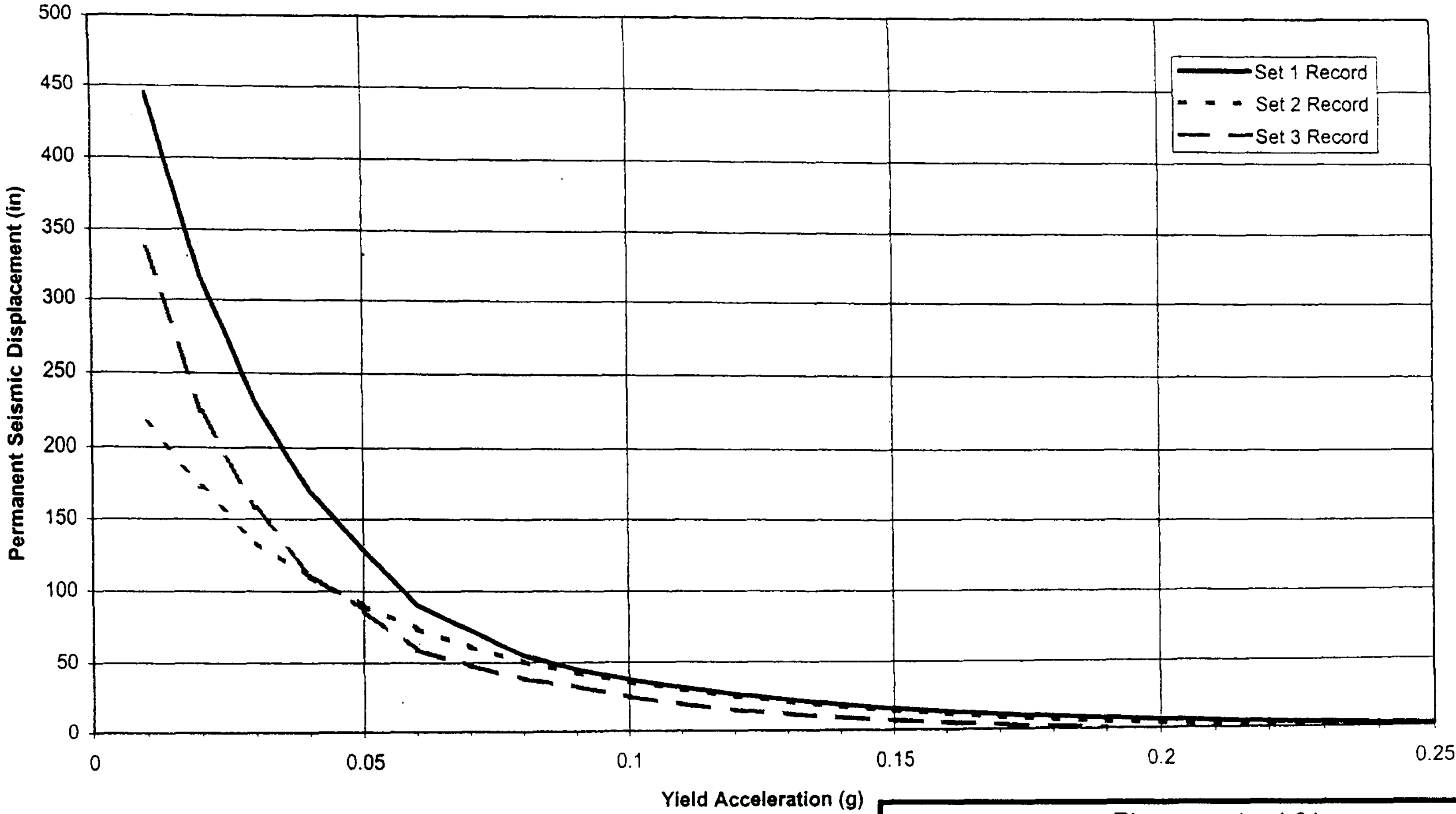

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-19b
SEISMICALLY INDUCED SLOPE DISPLACEMENTS VERSUS YIELD ACCELERATION (WEST SIDE OF IR SITE 2)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

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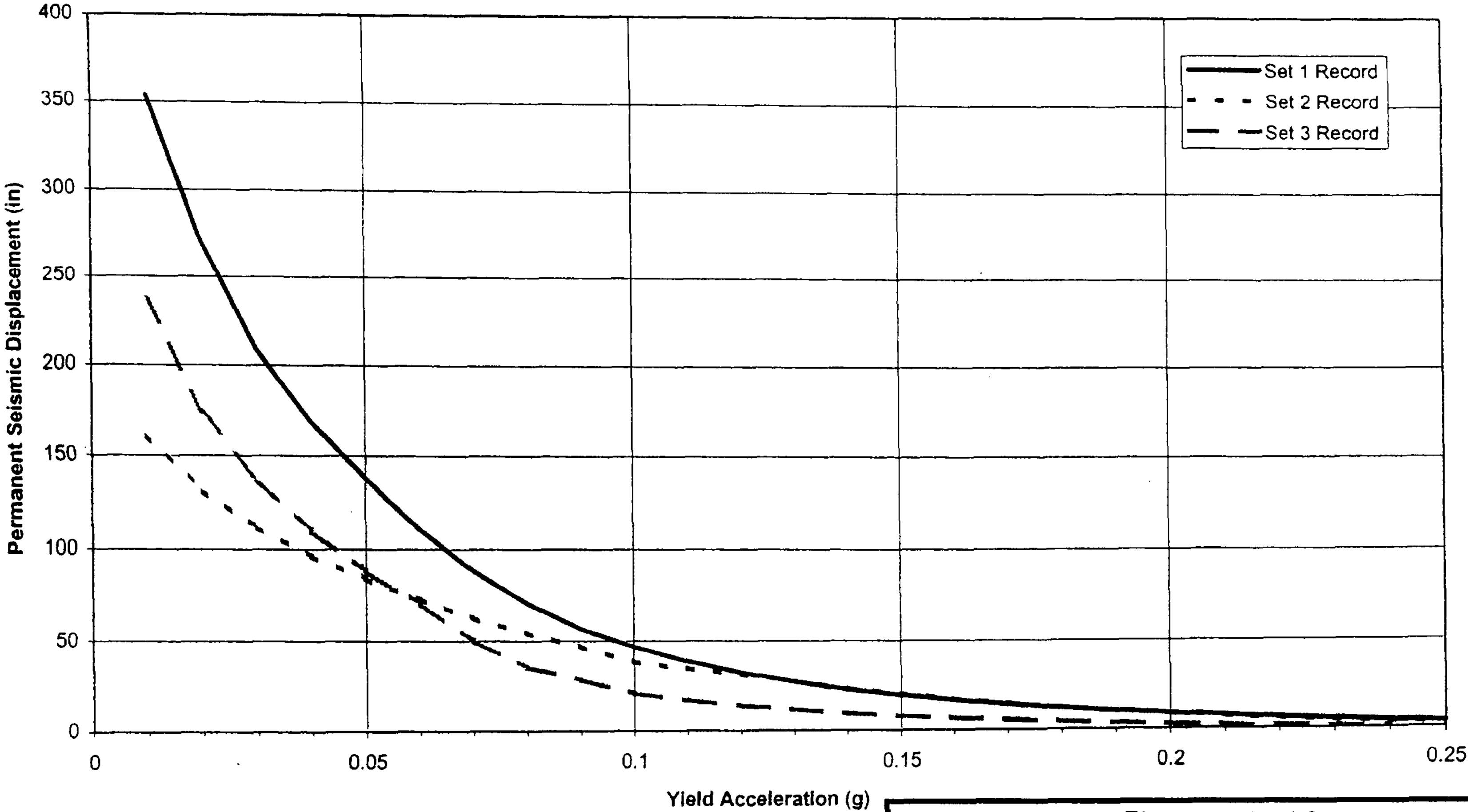

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-19c
SEISMICALLY INDUCED SLOPE DISPLACEMENTS VERSUS YIELD ACCELERATION (AREA BETWEEN IR SITES 1 AND 2)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER
ENVIRONMENTAL CORPORATION

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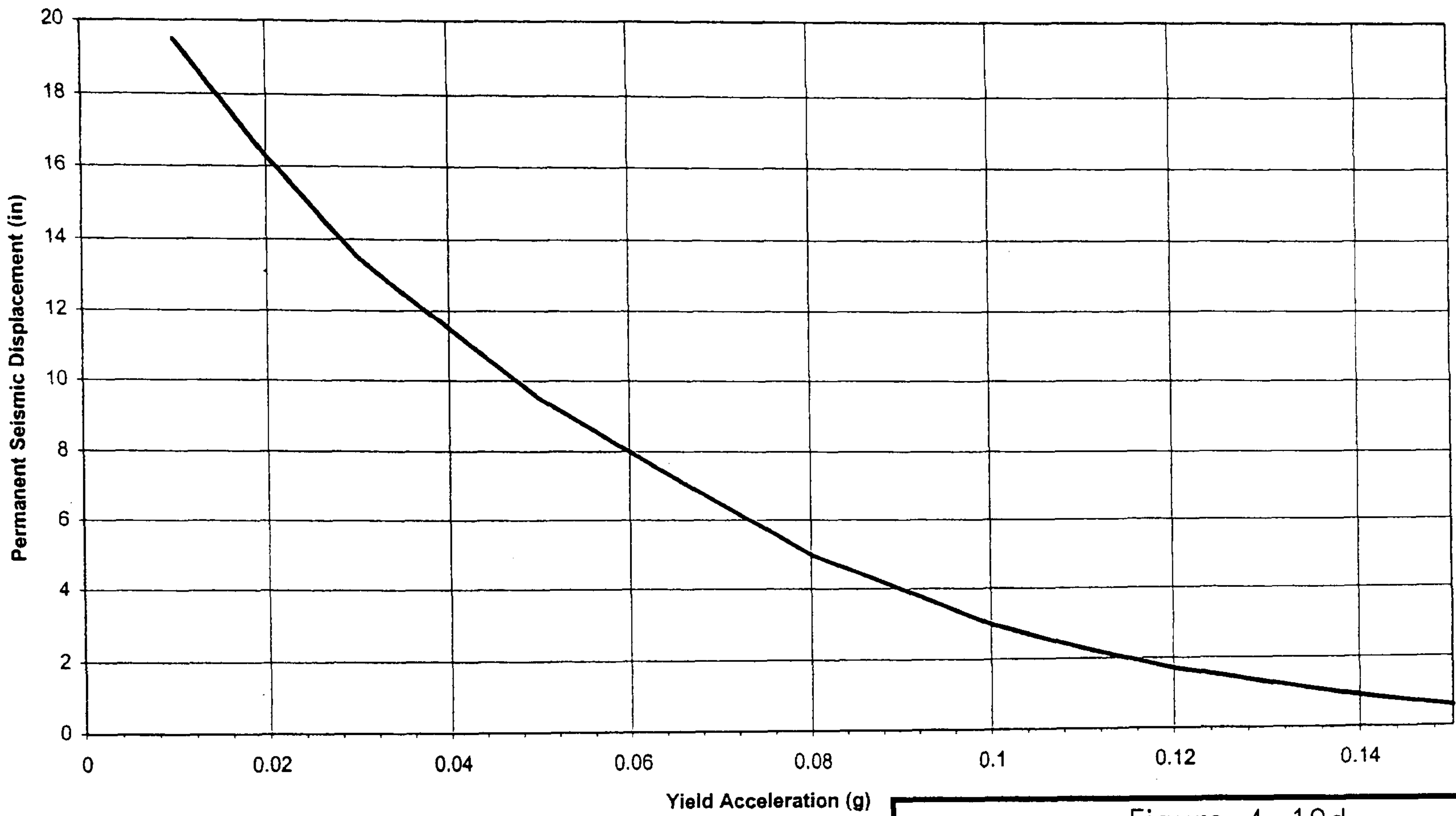

SOURCE: HUSHMAND ASSOCIATES, INC.

Figure 4-19d
SEISMICALLY INDUCED SLOPE DISPLACEMENTS VERSUS YIELD ACCELERATION (LOMA PRIETA EARTHQUAKE-ALAMEDA NAS HANGAR 23 RECORD)

Southwest Division
Naval Facilities Engineering Command

FOSTER WHEELER ENVIRONMENTAL CORPORATION

12 inches. Lateral deformation of liquefied soils was estimated to be larger than approximately 20 feet toward San Francisco Bay. Seismic slope instability was also identified as a major potential seismic hazard. The artificial fill is underlain by a relatively compressible and weak layer commonly known as Young Bay Mud. In addition to liquefaction-induced settlements from the upper fill material, soil sediments from the Young Bay Mud layer could experience approximately 4 to 6 inches of settlements due to liquefaction and consolidation. Therefore, the total seismically induced settlements could reach 18 inches (1.5 feet).

Seismic instability is mainly due to the weak Young Bay Mud layer. Post-earthquake static factors of safety calculated were greater than one for all the cross sections analyzed except for one case (Cross Section F-F'). Permanent slope deformations calculated ranged from 4 to 19 feet. The magnitude of the deformation is significant enough to trigger progressive failure in adjacent areas within and beyond the site boundary. In addition, seismic lateral displacements must be controlled to avoid release of the landfill waste to the San Francisco Bay in case of a failure from these displacements. Therefore, implementation of remedial measures appears necessary within and beyond the site boundaries to control seismically induced lateral slope displacements.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The procedures followed for the execution of ordnance and explosives waste (OEW) characterization work at Installation Restoration (IR) Site 2 concentrated on ensuring the safety of field personnel. Strict compliance with guidelines established for maximizing project quality control (QC) was maintained by the Project Quality Control Manager (PQCM). During the surface characterization of IR Site 2, one anti-tank/anti-personnel (AT/AP) inert land mine and one 20 millimeter (mm) target practice projectile were found. In addition to the surface characterization activities, a Time-Critical Removal Action (TCRA) was performed at the Possible OEW Burial Site, a 2.3-acre area located at the southern part of IR Site 2. A complete discussion of the TCRA is provided in a separate *Final Time-Critical Removal Action Closeout Report* [Foster Wheeler Environmental Corporation (FWENC), 2002a]. During the TCRA, 8,675 20mm target practice projectiles were uncovered. None of the OEW encountered contained any explosives or energetics. The AT/AP inert land mine was turned over to the Navy Explosive Ordnance Disposal (EOD) personnel. All of the target practice projectiles found were demilitarized and shipped to a Class III landfill facility for disposal as non-hazardous scrap steel.

Future remedial activities will include the placement of 4 feet of fill at IR Site 2 as a part of the presumptive remedy selected for the site. This landfill cap will act as the base of construction for use as a National Wildlife Refuge, with additional topsoil to be imported for site grading purposes. Land use controls will be established during the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) process including engineering and institutional controls that will address the landfill cap placement and any intrusive activities that will require excavation below the current land surface. A CERCLA Record of Decision will establish land use controls that will ensure that no invasive activities will disturb the existing landfill cap. The scope of work to be developed for placement of the landfill cap at IR Site 2 will require that only clean fill material be used for the cap construction. This will require appropriate screening mechanisms to ensure that fill material is free of hazardous, toxic, and radiological waste, including ordnance and explosives.

Upon completion of the surface characterization and TCRA at IR Site 2, the removal of the OEW on the site will be considered complete for the planned use of the land by the City of Alameda.

A geotechnical characterization of the site was performed in accordance with the requirements of the *Final Focused Remedial Investigation Work Plan* (Work Plan) (FWENC, 2002b). Field exploration consisted of performing 21 cone penetration tests (CPTs), excavating 21 test pits, and drilling 15 soil borings. Results of field exploration were used to evaluate the existing condition of cover soils and to identify seismic hazards at the site.

Thickness of the cover soil varied from 2 inches to 2 feet. The existing soil cover was found to be inconsistent, poorly compacted, and very permeable. Because of these conditions, the material was determined to be unsuitable for use as part of the final cover design.

The seismic hazards identified at IR Site 2 included liquefaction potential and seismic slope instability. An integrated CPT-based method (Robertson and Wride, 1997) was used to quantify the potential for liquefaction and to identify areas susceptible to liquefaction. Based on the analyses, the upper fill material at the site exhibited a high potential for liquefaction and was designated as liquefiable. Liquefaction-induced settlements are estimated to be up to 12 inches. The total expected seismic settlements from liquefaction of the upper fill material and liquefaction/consolidation of the Bay Sediments in the Young Bay Mud layer are approximately 18 inches (1.5 feet). Lateral deformation is estimated to be greater than approximately 20 feet.

Different cross sections at the site were analyzed for stability. The program, PC-STABL-5M (Achilleos, 1988), based on limit equilibrium theory, was used to obtain factors of safety against slope failure. Six different cross sections across IR Site 2 (Cross Sections C-C' to H-H') and one in the Additional Investigation Area (Cross Section I-I') were analyzed. Cross sections at IR Site 2 were analyzed with an assumed 4-foot-thick soil cover. Cross section I-I' is located on a former air strip where no future soil cover is planned. (Current state of practice in California requires a static factor of safety greater than 1.5.) All cross sections analyzed (except Cross Section C-C') had static factors of safety greater than 1.5. The factor of safety calculated for Cross Section C-C' was 1.46, less than the minimum required by the State of California. Therefore, remedial measures involving geotechnical improvements of existing site conditions are needed to increase the static factors of safety to meet the current standard of practice in California.

An extensive seismic hazard analysis was performed to obtain the peak horizontal ground acceleration (PHGA), design response spectrum, and acceleration time histories at the site. Using Newmark-type procedures, permanent lateral displacements at the site were obtained. Based on preliminary findings, predicted deformations are high, ranging from 4 to 19 feet. For post-earthquake stability conditions, the United States Army Corps of Engineers (USACE) recommends a post-earthquake static factor of safety greater than 1.0. This criterion was satisfied for all cross sections, except Cross Section F-F'.

In order to address the liquefaction potential concerns and other hazards such as seismically induced settlements and lateral displacements, a Feasibility Study will be conducted. The Feasibility Study will evaluate various alternatives to mitigate the geotechnical and seismic hazards identified in this report.

The alternatives will be based upon the following concepts:

- Increasing seismic stability in the site area by stabilizing and increasing the strength of the Young Bay Mud (Stratum II) by in situ mixing with cement or other admixture (for example, lime).
- Dredging and replacement of Young Bay Mud adjacent to the shoreline with stable quarry and rock fill materials.
- Installing stone columns by similar methods accelerating consolidation of Young Bay Mud by enhancing dissipation of excess pore pressures induced by earthquake.
- Minimizing lateral displacement and containing potential contaminants from leaking into the ocean by installing physical containment barriers along the shoreline (perimeter of the site).

Detailed Design Analyses

Further analyses using more sophisticated analytical/numerical methods will be required as part of the detailed design effort to evaluate and determine a range of more realistic potential seismically induced permanent deformation during the design earthquake. Criteria for acceptable deformation will be established in the Feasibility Study to evaluate technical performance of the selected alternatives.

Simplified analysis methods, such as Newmark method (Newmark, 1965) used for slope stability evaluations at IR Site 2, are a good approximation to estimate preliminary seismic deformations. However, these methods do not provide highly reliable estimates of seismically induced slope displacements, particularly for relatively large displacements such as those estimated for IR Site 2 and subsurface soils, which undergo partial loss of strength due to seismic loading. Therefore, detailed and comprehensive two- or three-dimensional dynamic numerical analysis methods should be used to provide a more realistic model of slope geometry and material properties as part of detailed evaluation of the site perimeter slopes during design of the selected remedial measure(s).

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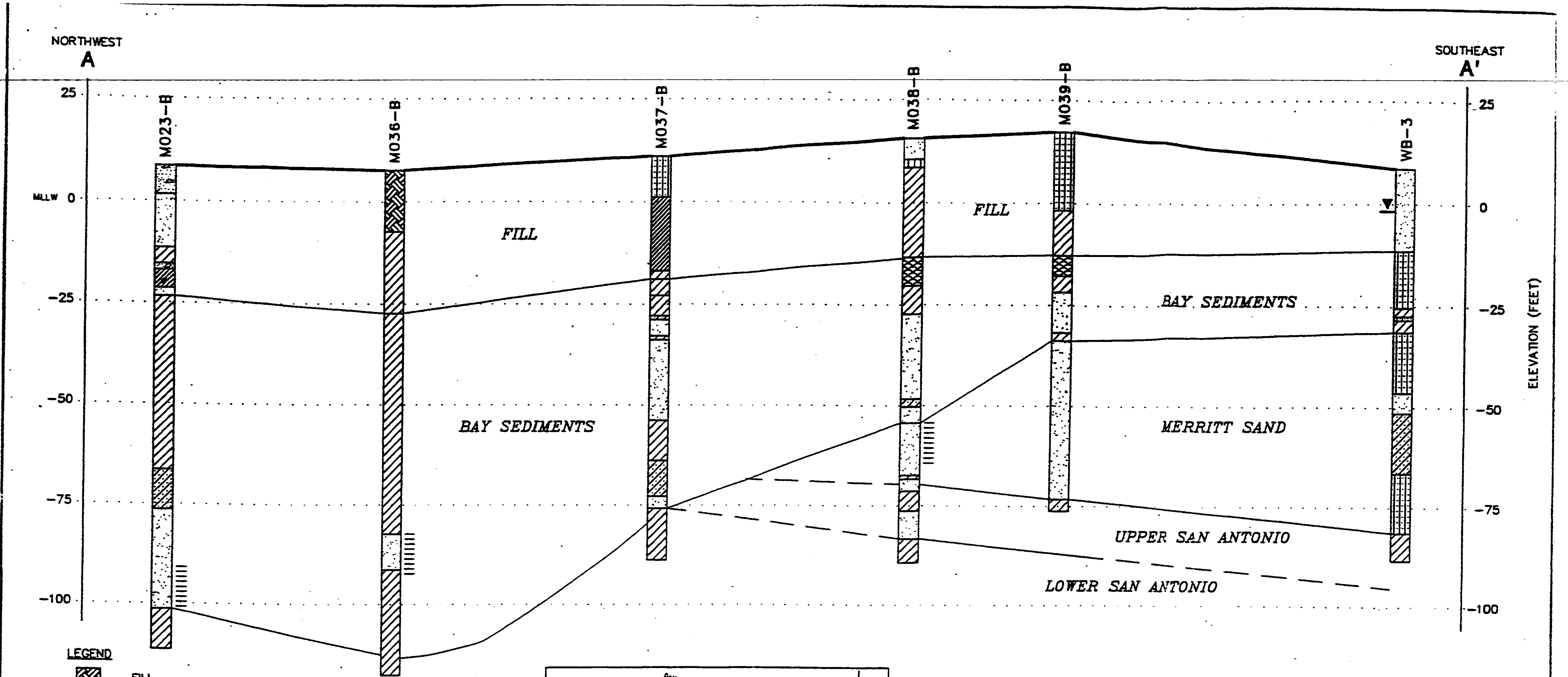
APPENDIX A

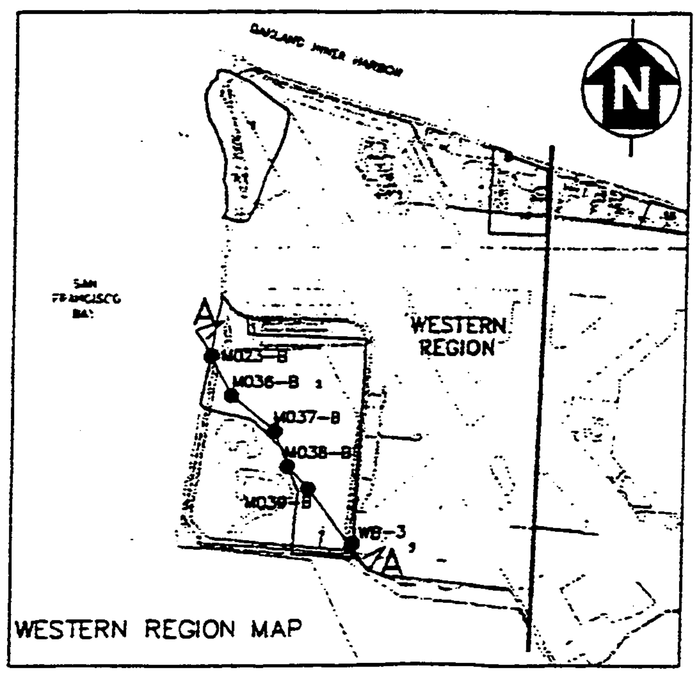
GEOLOGIC CROSS SECTIONS FROM PREVIOUS REMEDIAL INVESTIGATION REPORT

(Taken from TtEMI, 1999)



LEGEND

- FILL
- GW – WELL-GRADED GRAVELS
- GC – CLAYEY GRAVELS
- SP – POORLY-GRADED SANDS
- SM – SILTY SANDS
- SC – CLAYEY SANDS
- ML – SANDY SILTS, SILTY CLAYS
- CL – SANDY CLAYS, SILTY CLAYS
- CH – ORGANIC CLAYS, HIGH PLASTICITY
- SHELLS
- CONTACT
- INFERRED CONTACT
- APPROXIMATE WATER LEVEL MEASURED DURING DRILLING ACTIVITIES
- MONITORING WELL SCREENED INTERVAL
- MLLW MEAN LOWER LOW WATER



WESTERN REGION MAP

NOTE: GROUND SURFACE ELEVATIONS FOR WB-3 IS ESTIMATED.

VERTICAL EXAGGERATION IS 10X.

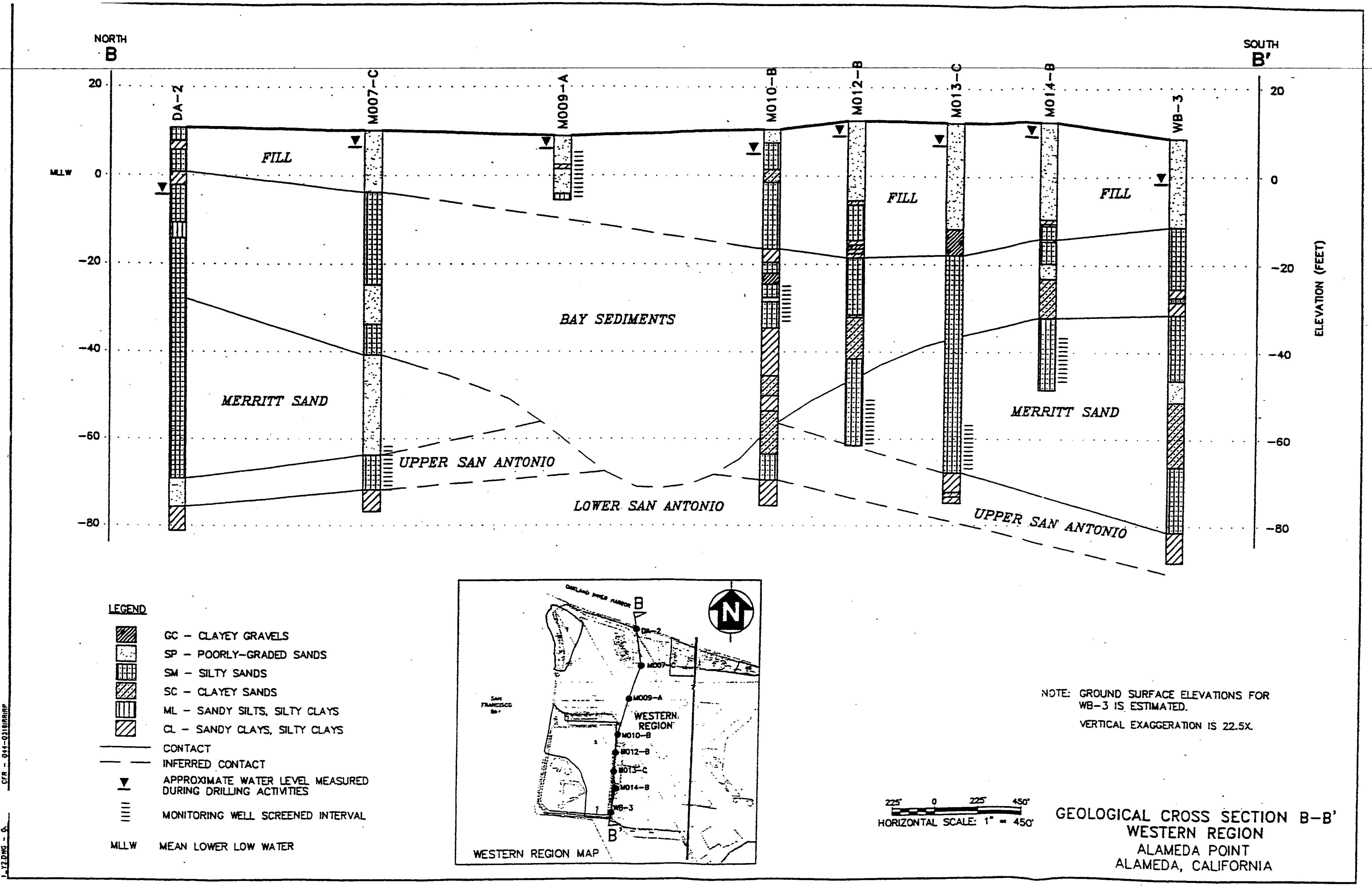
125' 0 125' 250'

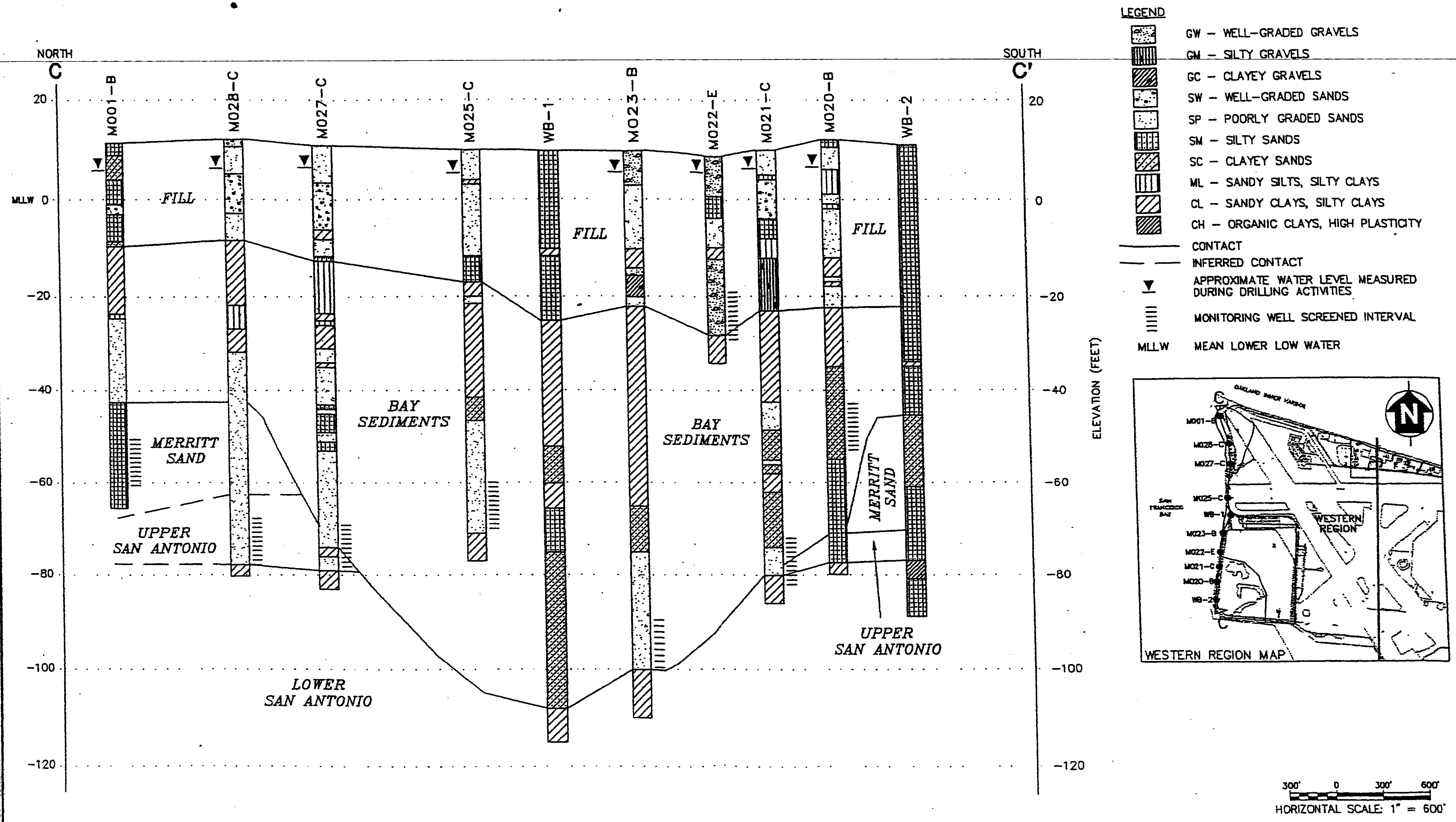
HORIZONTAL SCALE: 1" = 250'

GEOLOGICAL CROSS SECTION A–A'
WESTERN REGION
ALAMEDA POINT
ALAMEDA, CALIFORNIA

- CFR - 014-D316IRIRP

1-2-Z.Z.DWG - U..

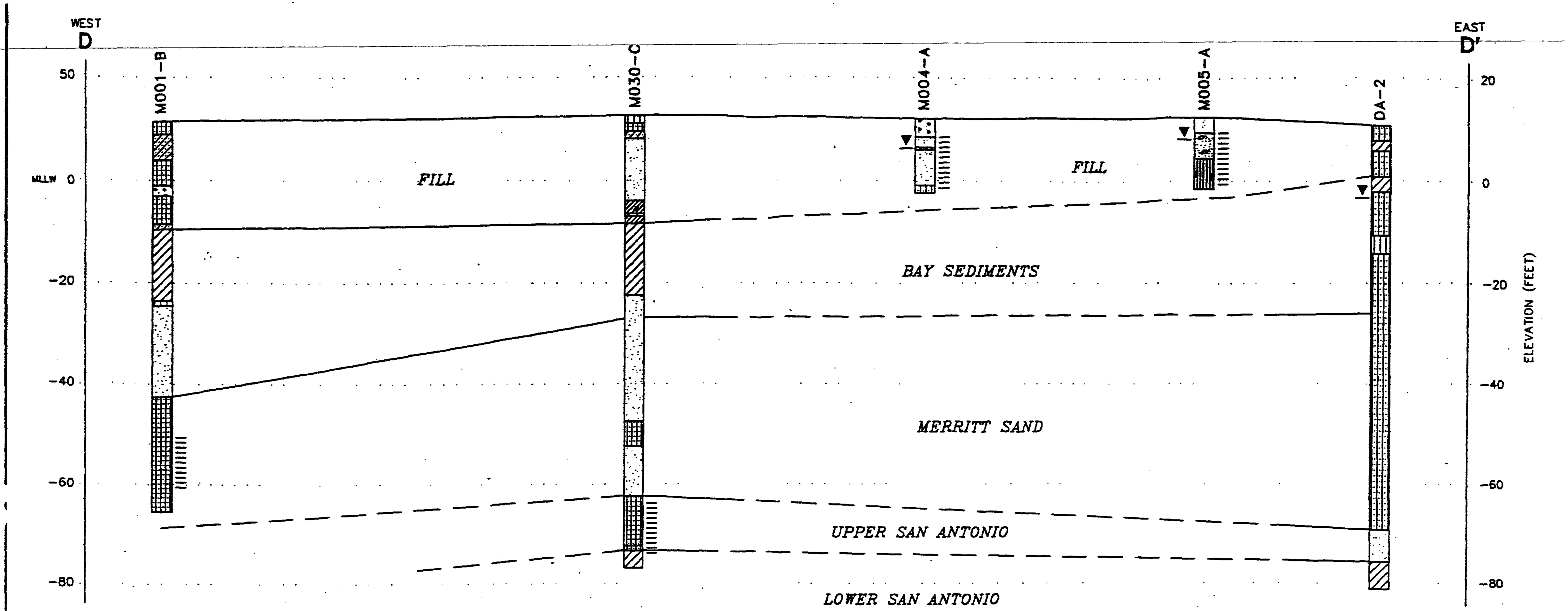

NORTH
B
SOUTH
B'
DA-2
M007-C
M009-A
M010-B
M012-B
M013-C
M014-B
WB-3
20
MLLW
0
-20
-40
-60
-80
ELEVATION (FEET)
FILL
BAY SEDIMENTS
MERRITT SAND
UPPER SAN ANTONIO
LOWER SAN ANTONIO
LEGEND
GC - CLAYEY GRAVELS
SP - POORLY-GRADED SANDS
SM - SILTY SANDS
SC - CLAYEY SANDS
ML - SANDY SILTS, SILTY CLAYS
CL - SANDY CLAYS, SILTY CLAYS
CONTACT
INFERRED CONTACT
APPROXIMATE WATER LEVEL MEASURED DURING DRILLING ACTIVITIES
MONITORING WELL SCREENED INTERVAL
MLLW MEAN LOWER LOW WATER
WESTERN REGION
WESTERN REGION MAP
N
NOTE: GROUND SURFACE ELEVATIONS FOR WB-3 IS ESTIMATED.
VERTICAL EXAGGERATION IS 22.5X.
225' 0 225' 450'
HORIZONTAL SCALE: 1" = 450'
GEOLOGICAL CROSS SECTION B-B'
WESTERN REGION
ALAMEDA POINT
ALAMEDA, CALIFORNIA



NOTE: GROUND SURFACE ELEVATIONS FOR WB-1 AND WB-2 ARE ESTIMATED.
VERTICAL EXAGGERATION IS 30X.

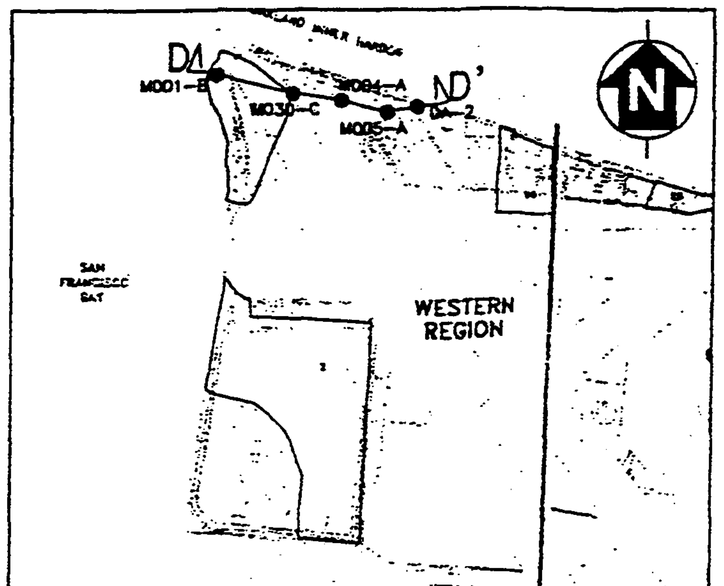
GEOLOGICAL CROSS SECTION C–C'
WESTERN REGION
ALAMEDA POINT
ALAMEDA, CALIFORNIA

I_YY.DWG - 05/23/97 - CFR - 044-0316RRIRP



LEGEND

- GW – WELL-GRADED GRAVELS
- GM – SILTY GRAVELS
- GC – CLAYEY GRAVELS
- SW – WELL-GRADED SANDS
- SP – POORLY-GRADED SANDS
- SM – SILTY SANDS
- SC – CLAYEY SANDS
- ML – SANDY SILTS, SILTY CLAYS
- CL – SANDY CLAYS, SILTY CLAYS
- CONTACT
- INFERRED CONTACT
- APPROXIMATE WATER LEVEL MEASURED DURING DRILLING ACTIVITIES
- MONITORING WELL SCREENED INTERVAL
- MLLW MEAN LOWER LOW WATER

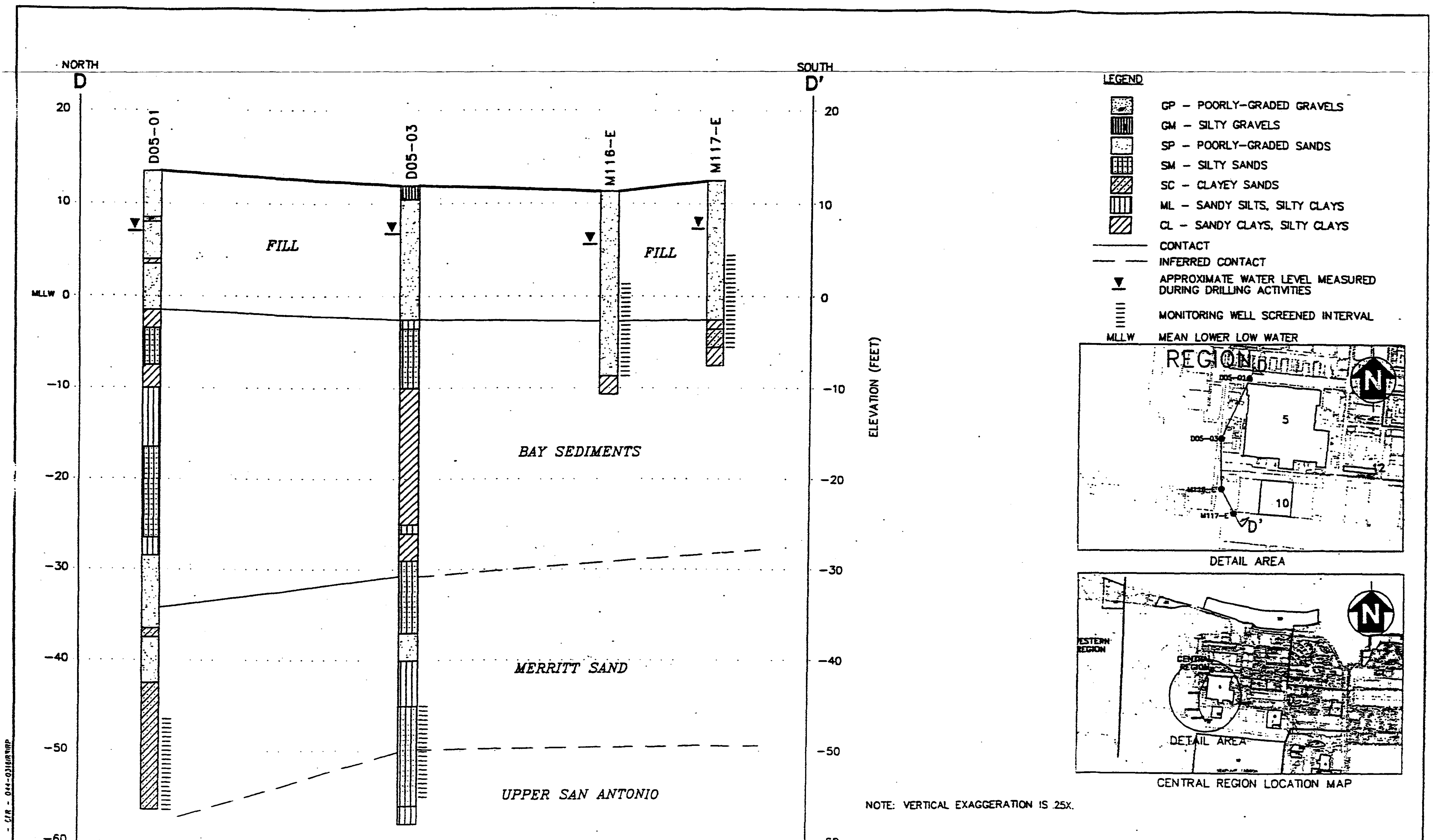


WESTERN REGION MAP

NOTE: GROUND SURFACE ELEVATIONS FOR DA-2 IS ESTIMATED.
VERTICAL EXAGGERATION IS 10X.

100' 0 100' 200'
HORIZONTAL SCALE: 1" = 200'

GEOLOGICAL CROSS SECTION D-D'
WESTERN REGION
ALAMEDA POINT
ALAMEDA, CALIFORNIA



GEOLOGICAL CROSS SECTION D-D'
CENTRAL REGION
ALAMEDA POINT
ALAMEDA, CALIFORNIA

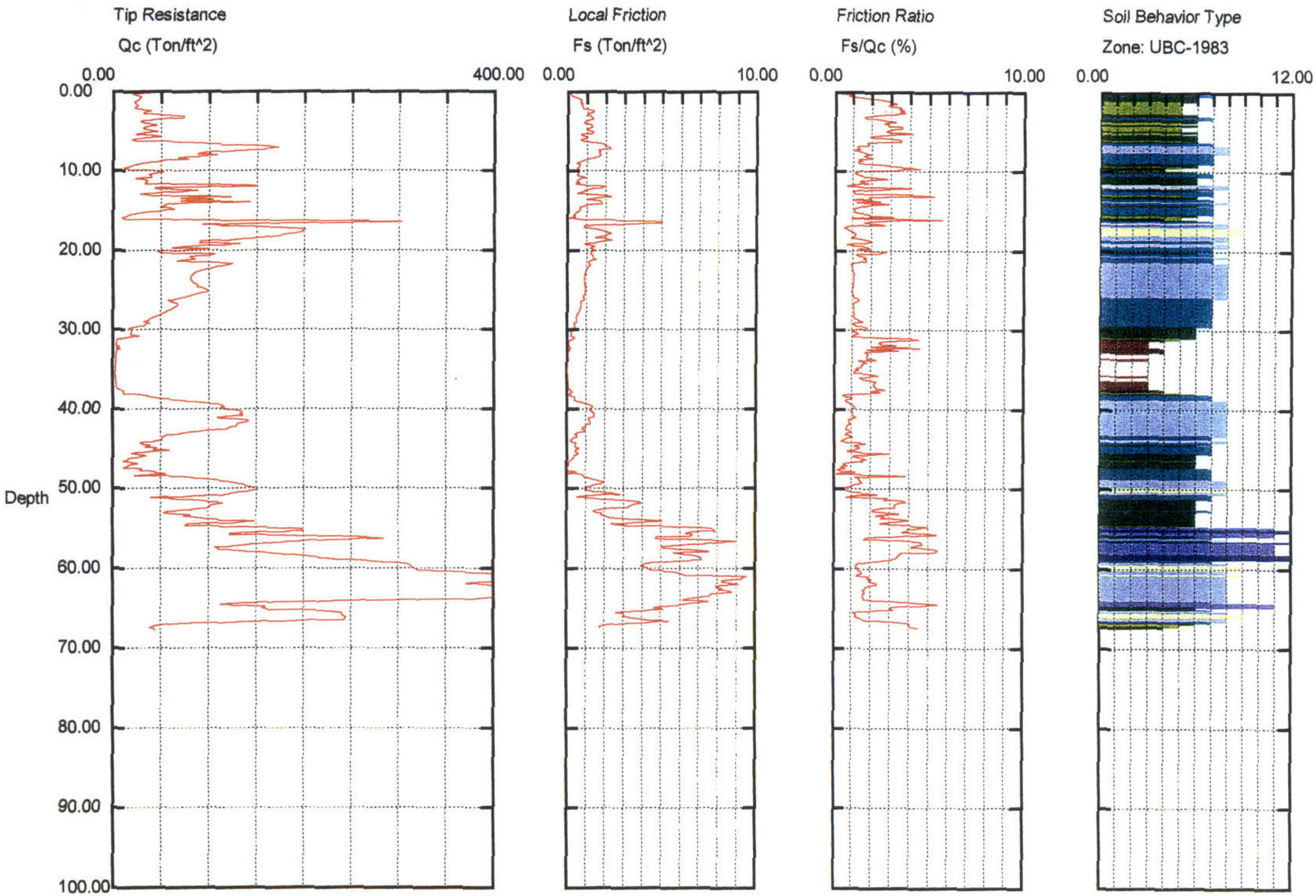
APPENDIX B

LOGS OF CPT SOUNDINGS AND SEISMIC VELOCITY MEASUREMENTS

Hushmand Associates

Operator: ALAMEDA NAS #2
Sounding: SDF120
Cone Used: 408/GO-VO/R#4

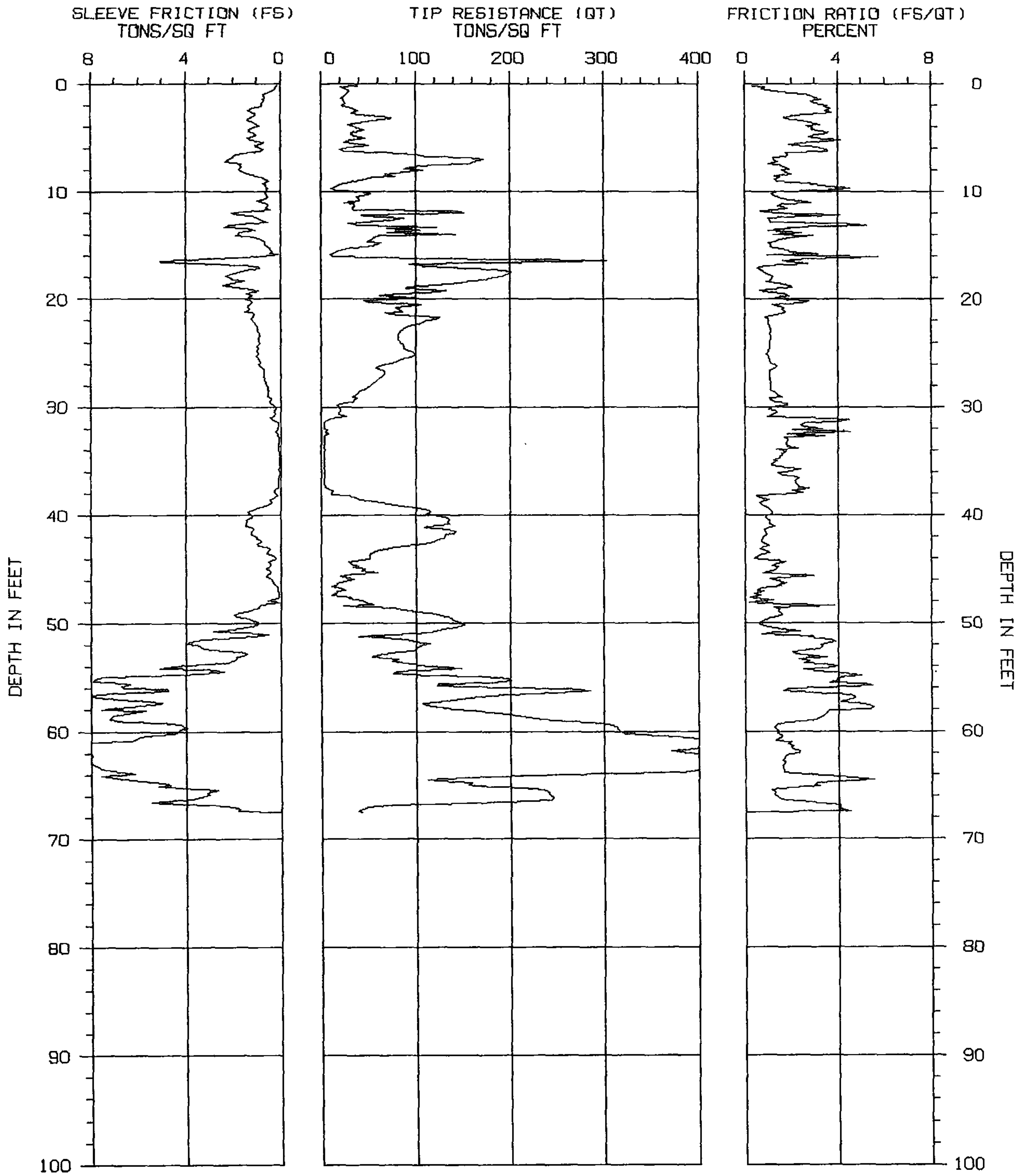
CPT Date/Time: 02-19-02 09:25
Location: CPT-01
Job Number: 010810



Maximum Depth = 67.75 feet

Depth Increment = 0.16 feet

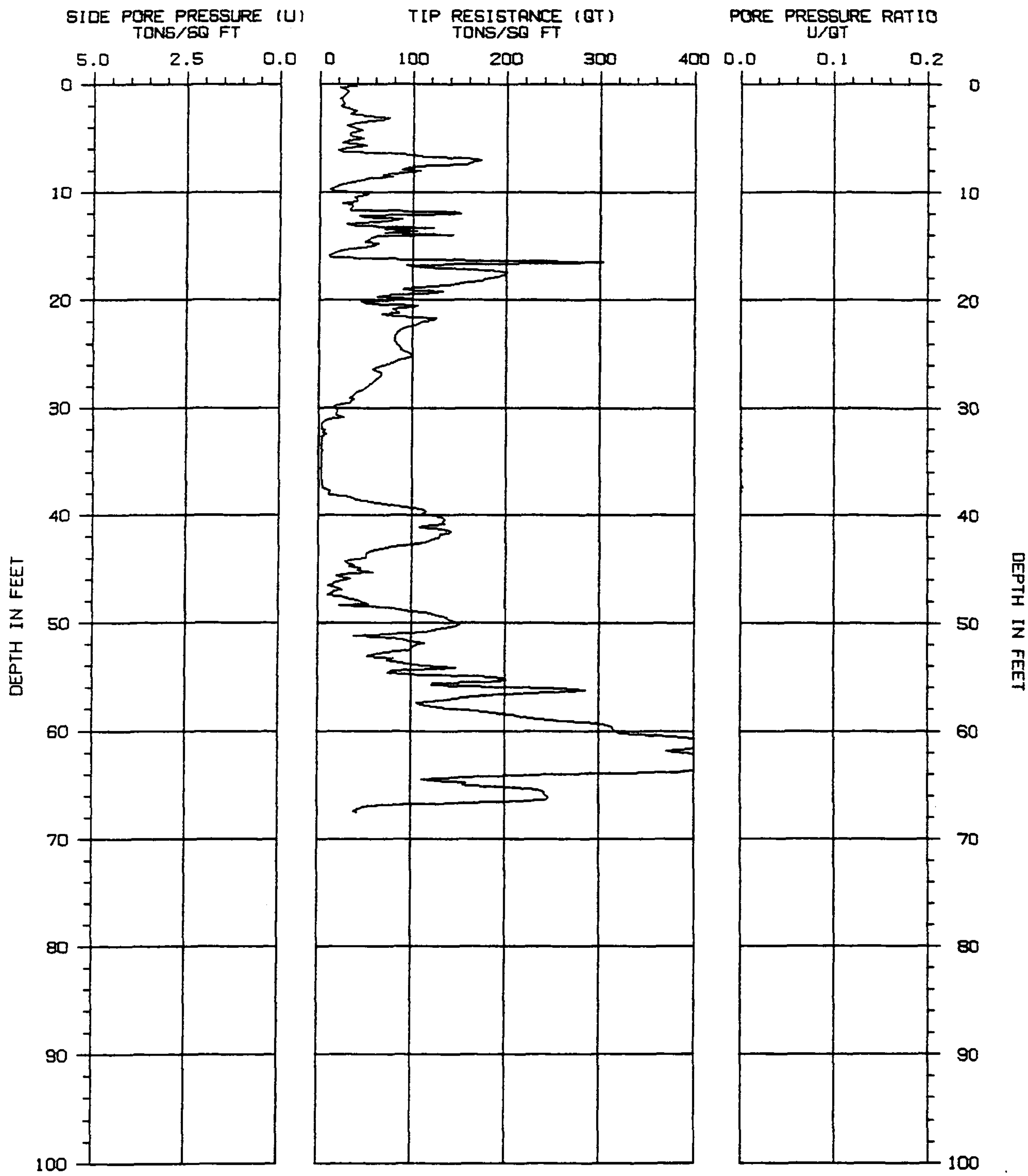
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)



TIP RESISTANCE CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST	SOUNDING NUMBER: CPT-01
PROJECT NAME : ALAMEDA NAS #2	CONE/RIG : 408/GO-VO/R#4
PROJECT NUMBER : 010810	DATE/TIME: 02-19-02 09:25

HFA



TIP RESISTANCE CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST	SOUNDING NUMBER: CPT-01
PROJECT NAME : ALAMEDA NAS #2	CONE/RIG : 408/GO-VO/R#4
PROJECT NUMBER : 010810	DATE/TIME: 02-19-02 09:25

HFA

CPT INTERPRETATIONS

SOUNDING : CPT-01 — PROJECT No.: 010810
PROJECT : ALAMEDA NAS #2 — CONE/RIG : 408/GO-VO/R#4
DATE/TIME: 02-19-02 09:25

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	25.49	.78	SILTY SAND to SANDY SILT	8	14	37		
.300	.98	26.98	2.63	CLAYEY SILT to SILTY CLAY	13	22		1.8	
.450	1.48	23.33	3.30	CLAYEY SILT to SILTY CLAY	12	19		1.5	
.600	1.97	22.52	3.55	CLAY to SILTY CLAY	15	24		1.5	
.750	2.46	39.69	3.48	CLAYEY SILT to SILTY CLAY	20	32		2.6	
.900	2.95	51.86	2.08	SANDY SILT to CLAYEY SILT	21	33		3.4	
1.050	3.44	55.22	2.48	SANDY SILT to CLAYEY SILT	22	35		3.7	
1.200	3.94	32.89	2.68	CLAYEY SILT to SILTY CLAY	16	26		2.2	
1.350	4.43	36.80	3.61	CLAYEY SILT to SILTY CLAY	18	29		2.4	
1.500	4.92	40.66	2.78	SANDY SILT to CLAYEY SILT	16	26		2.7	
1.650	5.41	24.58	2.89	CLAYEY SILT to SILTY CLAY	12	20		1.6	
1.800	5.91	29.87	3.15	CLAYEY SILT to SILTY CLAY	15	24		2.0	
1.950	6.40	68.54	1.79	SILTY SAND to SANDY SILT	23	37	66		43.5
2.100	6.89	156.38	1.27	SAND to SILTY SAND	39	62	89		46.5
2.250	7.38	157.91	1.03	SAND to SILTY SAND	39	61	89		46.0
2.400	7.87	88.00	1.94	SILTY SAND to SANDY SILT	29	44	73		43.5
2.550	8.37	67.71	2.02	SILTY SAND to SANDY SILT	23	33	65		42.0
2.700	8.86	47.18	1.29	SILTY SAND to SANDY SILT	16	22	55		39.5
2.850	9.35	20.99	3.00	CLAYEY SILT to SILTY CLAY	10	14		1.4	
3.000	9.84	17.27	3.36	CLAY to SILTY CLAY	12	15		1.1	
3.150	10.33	45.29	1.24	SILTY SAND to SANDY SILT	15	20	52		39.0
3.300	10.83	40.62	2.36	SANDY SILT to CLAYEY SILT	16	21		2.7	
3.450	11.32	36.46	1.43	SILTY SAND to SANDY SILT	12	15	44		38.0
3.600	11.81	142.40	.70	SAND	28	35	83		44.0
3.750	12.30	46.10	2.56	SANDY SILT to CLAYEY SILT	18	22		3.0	
3.900	12.80	46.91	1.13	SILTY SAND to SANDY SILT	16	18	50		38.0
4.050	13.29	122.69	1.90	SILTY SAND to SANDY SILT	41	47	77		42.5
4.200	13.78	70.64	2.46	SANDY SILT to CLAYEY SILT	28	32		4.7	
4.350	14.27	55.90	1.91	SILTY SAND to SANDY SILT	19	21	53		38.5
4.500	14.76	63.22	1.04	SILTY SAND to SANDY SILT	21	23	56		38.5
4.650	15.26	33.48	1.28	SILTY SAND to SANDY SILT	11	12	38		36.0
4.800	15.75	14.21	3.17	CLAY to SILTY CLAY	9	10		.9	
4.950	16.24	105.99	2.72	SANDY SILT to CLAYEY SILT	42	45		6.2	
5.100	16.73	93.82	2.73	SANDY SILT to CLAYEY SILT	38	39		5.5	
5.250	17.22	182.58	.68	SAND	37	38	85		43.5
5.400	17.72	196.81	1.09	SAND	39	41	87		44.0
5.550	18.21	173.97	1.14	SAND to SILTY SAND	43	45	84		43.5
5.700	18.70	122.22	1.96	SILTY SAND to SANDY SILT	41	42	73		42.0
5.850	19.19	132.31	.69	SAND	26	27	75		42.0
6.000	19.69	62.18	1.93	SILTY SAND to SANDY SILT	21	21	54		38.0
6.150	20.18	46.23	2.75	SANDY SILT to CLAYEY SILT	18	19		3.0	

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : CPT-01

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
6.300	20.67	100.21	1.36	SAND to SILTY SAND	25	25	67		40.0
6.450	21.16	85.32	1.76	SILTY SAND to SANDY SILT	28	28	62		39.0
6.600	21.65	125.64	.90	SAND to SILTY SAND	31	31	73		41.5
6.750	22.15	108.05	1.04	SAND to SILTY SAND	27	26	68		40.0
6.900	22.64	90.57	1.08	SAND to SILTY SAND	23	22	63		39.0
7.050	23.13	83.07	1.12	SAND to SILTY SAND	21	20	61		39.0
7.200	23.62	80.96	1.12	SAND to SILTY SAND	20	19	60		38.5
7.350	24.11	84.36	1.07	SAND to SILTY SAND	21	20	61		39.0
7.500	24.61	87.76	1.07	SAND to SILTY SAND	22	21	62		39.0
7.650	25.10	99.28	.96	SAND to SILTY SAND	25	23	65		39.5
7.800	25.59	84.66	1.07	SAND to SILTY SAND	21	20	60		38.5
7.950	26.08	69.68	1.32	SILTY SAND to SANDY SILT	23	22	55		38.0
8.100	26.57	63.71	1.10	SILTY SAND to SANDY SILT	21	20	52		38.0
8.250	27.07	65.56	1.08	SILTY SAND to SANDY SILT	22	20	52		38.0
8.400	27.56	59.55	1.09	SILTY SAND to SANDY SILT	20	18	50		37.5
8.550	28.05	51.99	1.12	SILTY SAND to SANDY SILT	17	16	46		36.5
8.700	28.54	41.32	1.21	SILTY SAND to SANDY SILT	14	12	39		35.5
8.850	29.04	33.18	1.63	SANDY SILT to CLAYEY SILT	13	12		2.5	
9.000	29.53	33.27	1.32	SILTY SAND to SANDY SILT	11	10	32		33.5
9.150	30.02	19.78	1.16	SANDY SILT to CLAYEY SILT	8	7		1.4	
9.300	30.51	18.80	1.38	SANDY SILT to CLAYEY SILT	8	7		1.4	
9.450	31.00	16.68	2.58	CLAYEY SILT to SILTY CLAY	8	7		1.0	
9.600	31.50	4.25	2.59	CLAY	4	4		.2	
9.750	31.99	3.91	3.32	CLAY	4	3		.2	
9.900	32.48	8.26	1.82	CLAYEY SILT to SILTY CLAY	4	4		.5	
10.050	32.97	3.82	1.83	SENSITIVE FINE GRAINED	2	2		.2	
10.200	33.46	3.91	1.79	SENSITIVE FINE GRAINED	2	2		.2	
10.350	33.96	5.10	1.37	SENSITIVE FINE GRAINED	3	2		.3	
10.500	34.45	3.70	1.62	SENSITIVE FINE GRAINED	2	2		.2	
10.650	34.94	3.99	1.25	SENSITIVE FINE GRAINED	2	2		.2	
10.800	35.43	3.44	1.16	SENSITIVE FINE GRAINED	2	1		.1	
10.950	35.93	3.59	1.95	SENSITIVE FINE GRAINED	2	2		.2	
11.100	36.42	3.89	2.06	CLAY	4	3		.1	
11.250	36.91	4.16	2.16	CLAY	4	3		.2	
11.400	37.40	4.72	2.33	CLAY	5	4		.2	
11.550	37.89	11.20	2.50	CLAYEY SILT to SILTY CLAY	6	5		.7	
11.700	38.39	36.92	.68	SILTY SAND to SANDY SILT	12	10	33		32.5
11.850	38.88	63.76	.64	SAND to SILTY SAND	16	13	48		36.5
12.000	39.37	102.34	.98	SAND to SILTY SAND	26	21	62		38.5
12.150	39.86	111.68	1.18	SAND to SILTY SAND	28	23	64		38.5
12.300	40.35	135.03	1.01	SAND to SILTY SAND	34	27	69		39.5
12.450	40.85	136.12	1.09	SAND to SILTY SAND	34	27	70		39.5
12.600	41.34	134.86	.85	SAND to SILTY SAND	34	27	69		39.0
12.750	41.83	131.02	.96	SAND to SILTY SAND	33	26	68		39.0
12.900	42.32	122.05	.71	SAND to SILTY SAND	31	24	66		39.0
13.050	42.81	90.67	1.01	SAND to SILTY SAND	23	18	57		38.0
13.200	43.31	57.49	.96	SILTY SAND to SANDY SILT	19	15	44		36.0
13.350	43.80	51.35	.55	SAND to SILTY SAND	13	10	41		35.0
13.500	44.29	28.91	1.76	SANDY SILT to CLAYEY SILT	12	9		2.1	
13.650	44.78	34.06	1.44	SANDY SILT to CLAYEY SILT	14	11		2.5	

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

SOUNDING : CPT-01

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
13.800	45.28	59.08	.76	SAND to SILTY SAND	15	11	45		36.0
13.950	45.77	27.62	1.59	SANDY SILT to CLAYEY SILT	11	9		2.0	
14.100	46.26	17.51	1.77	CLAYEY SILT to SILTY CLAY	9	7		1.2	
14.250	46.75	20.56	.68	SANDY SILT to CLAYEY SILT	8	6		1.8	
14.400	47.24	16.10	.43	SANDY SILT to CLAYEY SILT	6	5		1.3	
14.550	47.74	28.91	.48	SILTY SAND to SANDY SILT	10	7	24		30.5
14.700	48.23	55.11	1.52	SILTY SAND to SANDY SILT	18	14	42		35.0
14.850	48.72	85.66	1.46	SILTY SAND to SANDY SILT	29	22	55		37.0
15.000	49.21	123.77	1.59	SAND to SILTY SAND	31	23	65		38.5
15.150	49.70	139.24	.90	SAND to SILTY SAND	35	26	68		39.0
15.300	50.20	150.67	.71	SAND	30	23	70		39.0
15.450	50.69	120.59	2.36	SILTY SAND to SANDY SILT	40	30	64		38.5
15.600	51.18	39.45	2.89	CLAYEY SILT to SILTY CLAY	20	15		2.4	
15.750	51.67	99.60	3.86	CLAYEY SILT to SILTY CLAY	50	37		5.7	
15.900	52.17	102.04	3.28	SANDY SILT to CLAYEY SILT	41	30		5.8	
16.050	52.66	81.77	2.07	SILTY SAND to SANDY SILT	27	20	52		37.0
16.200	53.15	53.66	3.50	CLAYEY SILT to SILTY CLAY	27	20		3.0	
16.350	53.64	88.21	2.56	SANDY SILT to CLAYEY SILT	35	26		5.7	
16.500	54.13	148.27	3.43	SANDY SILT to CLAYEY SILT	59	43		8.5	
16.650	54.63	75.25	4.29	CLAYEY SILT to SILTY CLAY	38	27		4.2	
16.800	55.12	197.70	3.96	*SAND to CLAYEY SAND	99	71			
16.950	55.61	123.09	5.16	*VERY STIFF FINE GRAINED	100	89			
17.100	56.10	258.53	1.85	SAND to SILTY SAND	65	46	85		42.0
17.250	56.59	196.28	4.56	*VERY STIFF FINE GRAINED	100	100			
17.400	57.09	138.83	4.42	*VERY STIFF FINE GRAINED	100	99			
17.550	57.58	109.96	5.36	*VERY STIFF FINE GRAINED	100	78			
17.700	58.07	157.91	3.60	SANDY SILT to CLAYEY SILT	63	45		9.1	
17.850	58.56	207.31	3.41	*SAND to CLAYEY SAND	100	73			
18.000	59.06	262.01	2.48	SILTY SAND to SANDY SILT	87	61	84		42.0
18.150	59.55	312.43	1.29	SAND	62	44	89		42.5
18.300	60.04	317.04	1.37	SAND	63	44	90		42.5
18.450	60.53	377.52	1.58	SAND to SILTY SAND	94	66	95		43.5
18.600	61.02	489.86	1.95	SAND to SILTY SAND	100	85	100		44.5
18.750	61.52	421.75	1.96	SAND to SILTY SAND	100	73	98		44.0
18.900	62.01	389.65	2.34	SAND to SILTY SAND	97	67	95		43.5
19.050	62.50	482.75	1.71	SAND to SILTY SAND	100	83	100		44.0
19.200	62.99	509.07	1.72	SAND to SILTY SAND	100	87	100		44.5
19.350	63.48	467.68	1.62	SAND to SILTY SAND	100	80	100		44.0
19.500	63.98	302.42	2.04	SAND to SILTY SAND	76	52	88		42.0
19.650	64.47	112.22	5.52	*VERY STIFF FINE GRAINED	100	76			
19.800	64.96	156.11	2.99	SANDY SILT to CLAYEY SILT	62	42		9.0	
19.950	65.45	234.44	1.13	SAND	47	32	80		40.5
20.100	65.94	242.51	1.25	SAND	49	33	81		40.5
20.250	66.44	243.06	1.78	SAND to SILTY SAND	61	41	81		40.5
20.400	66.93	68.88	4.07	CLAYEY SILT to SILTY CLAY	34	23		3.8	
20.550	67.42	39.26	4.51	CLAY to SILTY CLAY	26	17		2.1	

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

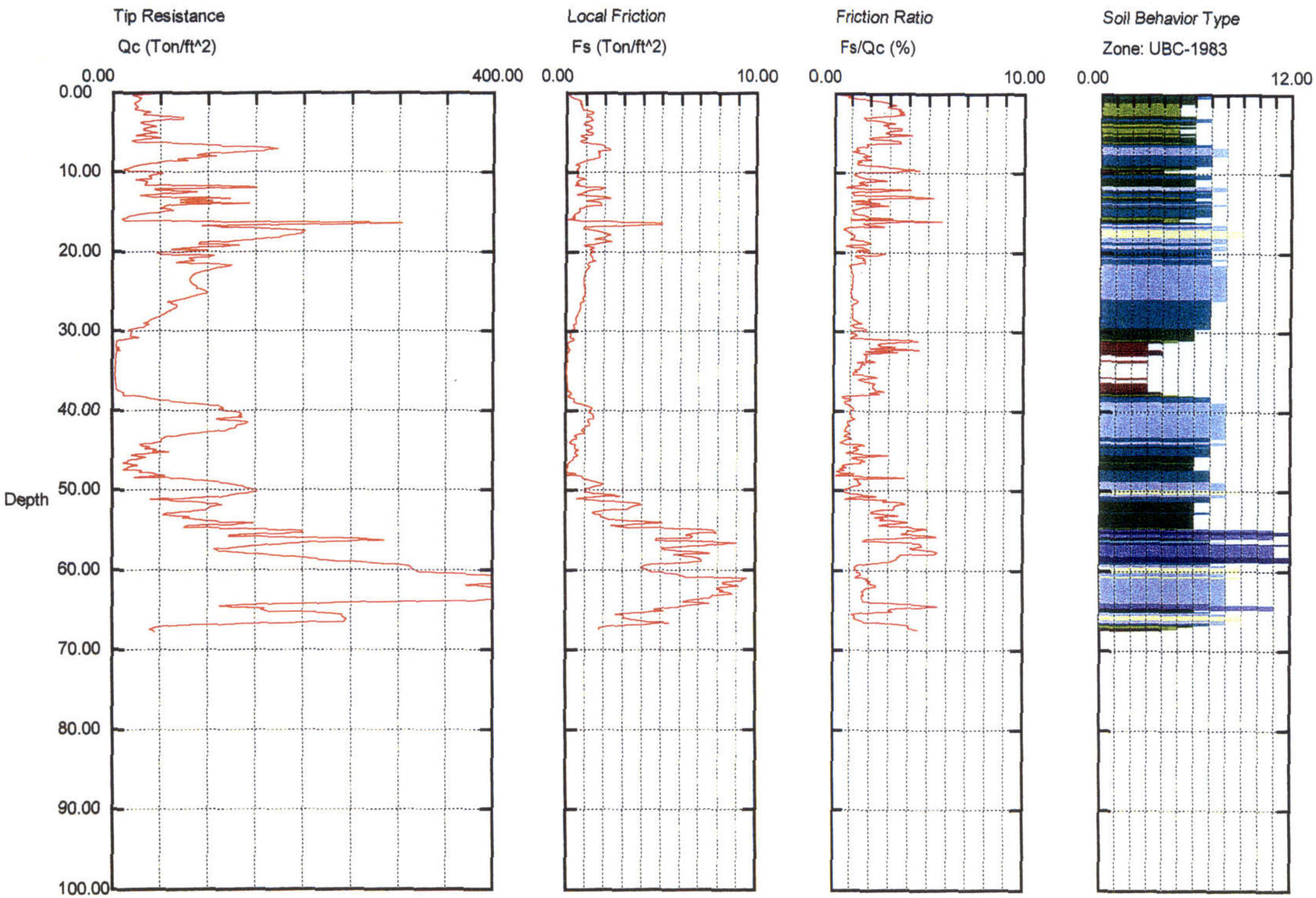
HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

Hushmand Associates

Operator: ALAMEDA NAS #2
Sounding: SDF120
Cone Used: 408/GO-VO/R#4

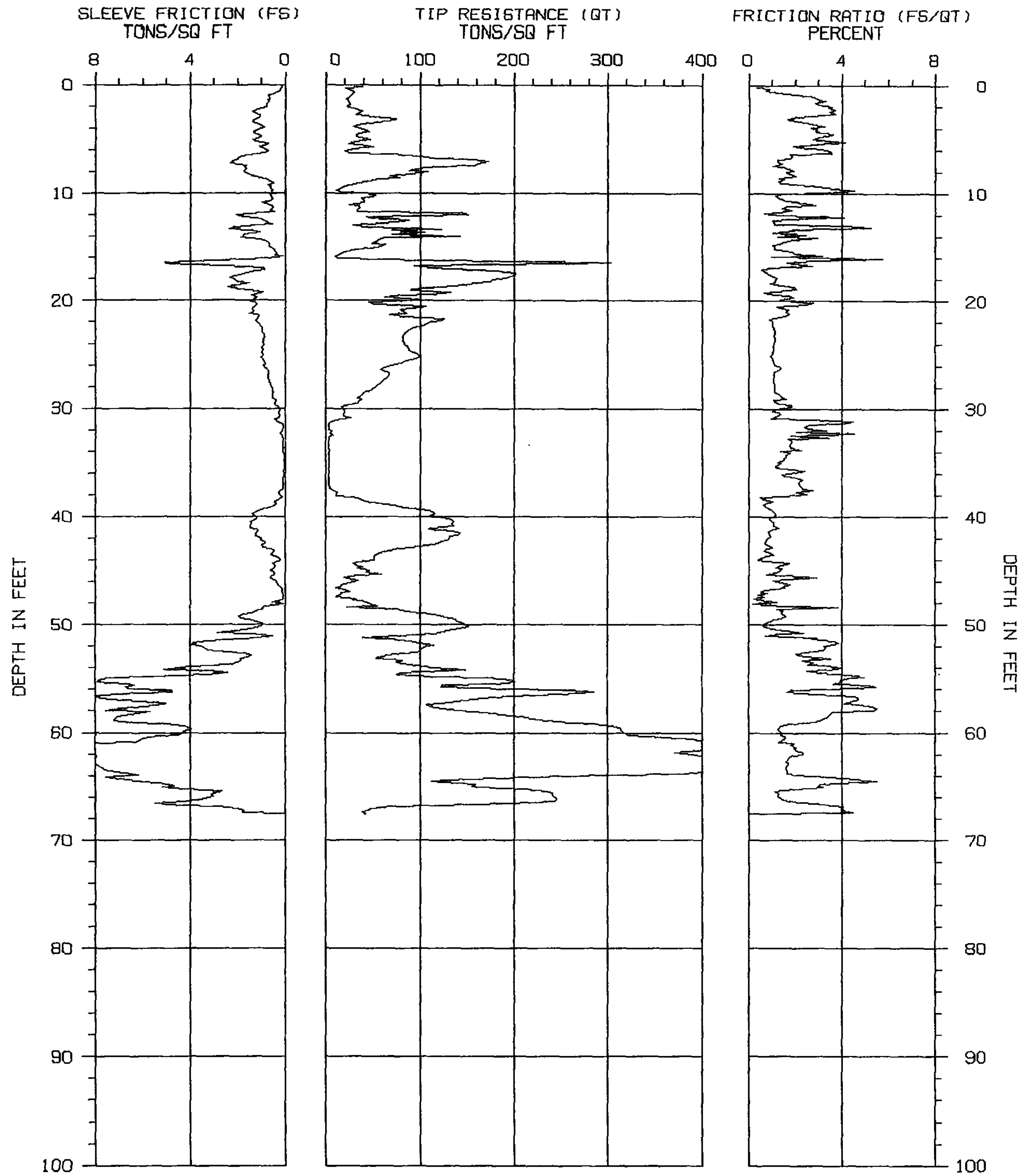
CPT Date/Time: 02-19-02 09:25
Location: CPT-01
Job Number: 010810



Maximum Depth = 67.75 feet

Depth Increment = 0.16 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)



TIP RESISTANCE CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST — SOUNDING NUMBER: CPT-01

PROJECT NAME : ALAMEDA NAS #2	CONE/RIG : 408/GO-VO/R#4
PROJECT NUMBER : 010810	DATE/TIME: 02-19-02 09:25

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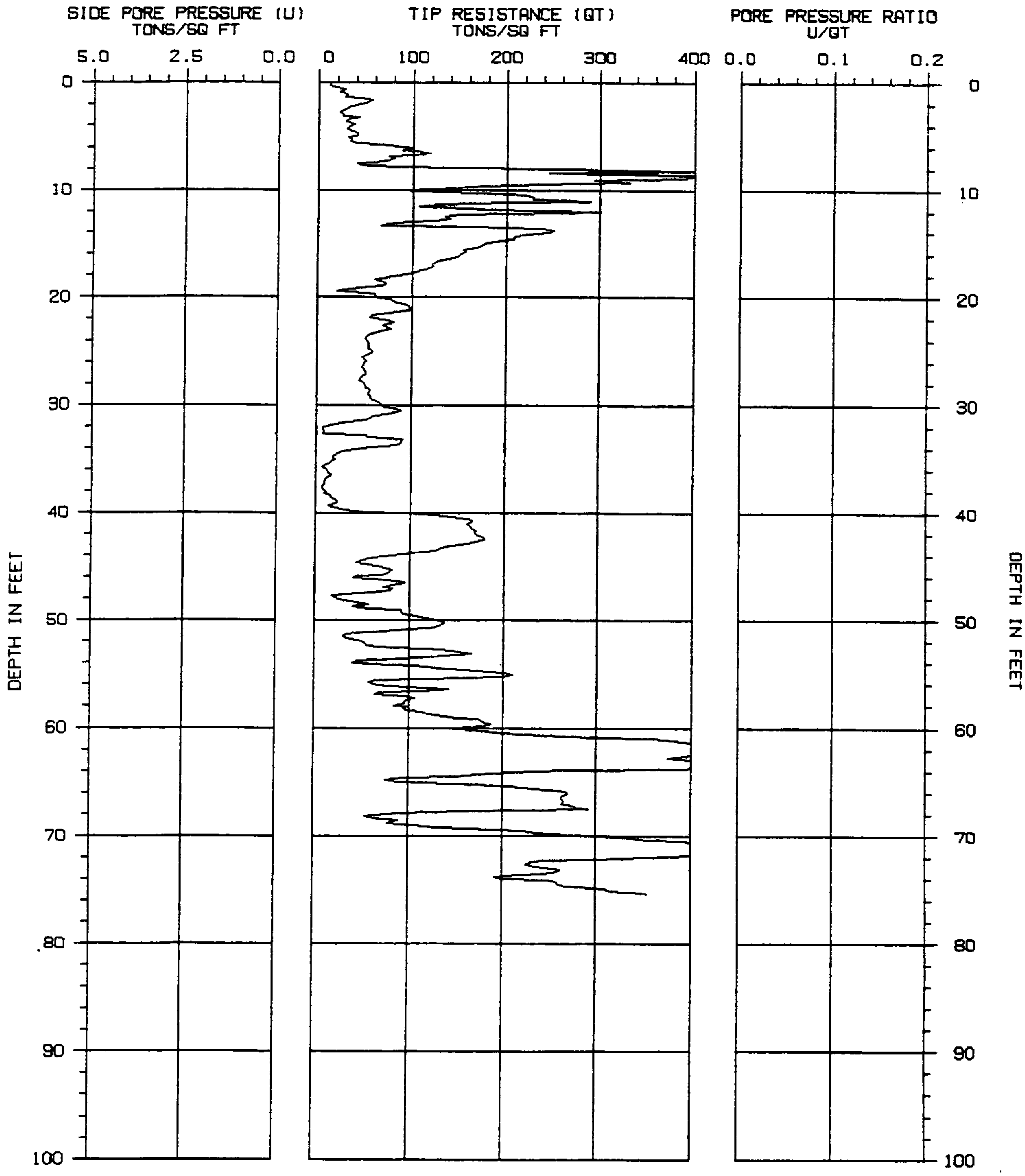

TIP RESISTANCE CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST	SOUNDING NUMBER: CPT-02
PROJECT NAME : ALAMEDA NAS #2	CONE/RIG : 408/GO-VO/R#4
PROJECT NUMBER : 010810	DATE/TIME: 02-19-02 10:49

HFA

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*                        CPT INTERPRETATIONS                          *
*                                                                     *
*  SOUNDING : CPT-02                    PROJECT No.: 010810           *
*  PROJECT  : ALAMEDA NAS #2            CONE/RIG : 408/GO-VO/R#4      *
*  DATE/TIME: 02-19-02 10:49                                          *
*                                                                     *
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DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	19.48	1.54	SANDY SILT to CLAYEY SILT	8	12		1.6	
.300	.98	26.81	2.20	SANDY SILT to CLAYEY SILT	11	17		1.8	
.450	1.48	43.13	1.34	SILTY SAND to SANDY SILT	14	23	52		47.5
.600	1.97	45.00	1.91	SANDY SILT to CLAYEY SILT	18	29		3.0	
.750	2.46	29.68	2.02	SANDY SILT to CLAYEY SILT	12	19		2.4	
.900	2.95	25.62	1.17	SANDY SILT to CLAYEY SILT	10	16		2.0	
1.050	3.44	31.31	1.63	SANDY SILT to CLAYEY SILT	13	20		2.5	
1.200	3.94	37.65	1.97	SANDY SILT to CLAYEY SILT	15	24		2.5	
1.350	4.43	33.59	1.52	SANDY SILT to CLAYEY SILT	13	21		2.7	
1.500	4.92	40.43	1.76	SANDY SILT to CLAYEY SILT	16	26		3.2	
1.650	5.41	34.01	1.94	SANDY SILT to CLAYEY SILT	14	22		2.7	
1.800	5.91	82.66	1.79	SILTY SAND to SANDY SILT	28	44	71		44.5
1.950	6.40	103.82	1.84	SILTY SAND to SANDY SILT	35	55	77		45.0
2.100	6.89	75.04	2.05	SILTY SAND to SANDY SILT	25	40	68		43.5
2.250	7.38	58.78	2.01	SILTY SAND to SANDY SILT	20	30	61		42.0
2.400	7.87	136.88	.77	SAND	27	41	85		45.5
2.550	8.37	245.06	1.18	SAND	49	71	100		47.5
2.700	8.86	389.03	1.12	SAND	78	100	100		49.0
2.850	9.35	276.14	1.30	SAND	55	75	100		47.5
3.000	9.84	126.51	1.79	SILTY SAND to SANDY SILT	42	56	82		44.0
3.150	10.33	199.91	.99	SAND	40	52	94		46.0
3.300	10.83	227.68	3.32	*SAND to CLAYEY SAND	100	100			
3.450	11.32	129.23	1.24	SAND to SILTY SAND	32	40	80		43.5
3.600	11.81	175.18	1.50	SAND to SILTY SAND	44	53	89		44.5
3.750	12.30	149.63	1.98	SILTY SAND to SANDY SILT	50	59	83		44.0
3.900	12.80	124.66	1.67	SILTY SAND to SANDY SILT	42	48	78		43.0
4.050	13.29	66.88	2.39	SANDY SILT to CLAYEY SILT	27	31		4.4	
4.200	13.78	251.22	.97	SAND	50	56	97		45.5
4.350	14.27	217.06	.96	SAND	43	48	92		45.0
4.500	14.76	195.32	.53	SAND	39	42	89		44.5
4.650	15.26	173.53	.66	SAND	35	37	85		43.5
4.800	15.75	156.57	.71	SAND	31	33	82		43.0
4.950	16.24	147.42	.80	SAND	29	31	80		43.0
5.100	16.73	124.77	.87	SAND to SILTY SAND	31	33	75		42.0
5.250	17.22	119.76	.90	SAND to SILTY SAND	30	31	73		42.0
5.400	17.72	104.46	.82	SAND to SILTY SAND	26	27	69		40.5
5.550	18.21	68.88	1.06	SILTY SAND to SANDY SILT	23	24	57		38.5
5.700	18.70	72.42	.58	SAND to SILTY SAND	18	18	58		38.5
5.850	19.19	35.76	2.01	SANDY SILT to CLAYEY SILT	14	15		2.3	
6.000	19.69	60.87	.61	SAND to SILTY SAND	15	15	53		38.0
6.150	20.18	76.95	.75	SAND to SILTY SAND	19	19	59		39.0

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : CPT-02

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
6.300	20.67	90.78	.80	SAND to SILTY SAND	23	23	64		39.5
6.450	21.16	98.30	.85	SAND to SILTY SAND	25	24	66		39.5
6.600	21.65	57.93	1.29	SILTY SAND to SANDY SILT	19	19	51		38.0
6.750	22.15	74.99	.68	SAND to SILTY SAND	19	18	58		38.5
6.900	22.64	69.51	.92	SAND to SILTY SAND	17	17	56		38.5
7.050	23.13	68.77	.97	SAND to SILTY SAND	17	17	55		38.0
7.200	23.62	52.07	.90	SILTY SAND to SANDY SILT	17	17	47		37.0
7.350	24.11	53.58	.88	SILTY SAND to SANDY SILT	18	17	48		37.0
7.500	24.61	54.22	.87	SILTY SAND to SANDY SILT	18	17	48		37.0
7.650	25.10	59.04	.85	SAND to SILTY SAND	15	14	50		37.5
7.800	25.59	48.18	.93	SILTY SAND to SANDY SILT	16	15	44		36.5
7.950	26.08	50.65	.95	SILTY SAND to SANDY SILT	17	16	45		36.5
8.100	26.57	48.84	.84	SILTY SAND to SANDY SILT	16	15	44		36.5
8.250	27.07	51.86	.93	SILTY SAND to SANDY SILT	17	16	46		36.5
8.400	27.56	45.72	1.05	SILTY SAND to SANDY SILT	15	14	42		36.0
8.550	28.05	50.50	.97	SILTY SAND to SANDY SILT	17	15	45		36.5
8.700	28.54	55.41	.90	SILTY SAND to SANDY SILT	18	17	47		37.0
8.850	29.04	55.02	.95	SILTY SAND to SANDY SILT	18	17	47		37.0
9.000	29.53	59.40	.93	SILTY SAND to SANDY SILT	20	18	49		37.0
9.150	30.02	68.81	.93	SAND to SILTY SAND	17	15	53		38.0
9.300	30.51	89.04	.75	SAND to SILTY SAND	22	20	60		38.5
9.450	31.00	62.50	.82	SAND to SILTY SAND	16	14	50		37.0
9.600	31.50	43.81	1.55	SILTY SAND to SANDY SILT	15	13	40		35.5
9.750	31.99	9.58	2.19	CLAYEY SILT to SILTY CLAY	5	4		.6	
9.900	32.48	7.80	1.41	CLAYEY SILT to SILTY CLAY	4	3		.5	
10.050	32.97	54.64	1.17	SILTY SAND to SANDY SILT	18	16	45		36.5
10.200	33.46	89.40	.88	SAND to SILTY SAND	22	19	59		38.5
10.350	33.96	57.17	1.07	SILTY SAND to SANDY SILT	19	16	47		36.5
10.500	34.45	23.20	1.12	SANDY SILT to CLAYEY SILT	9	8		1.7	
10.650	34.94	18.14	.94	SANDY SILT to CLAYEY SILT	7	6		1.3	
10.800	35.43	13.28	1.43	CLAYEY SILT to SILTY CLAY	7	6		.9	
10.950	35.93	10.56	.85	CLAYEY SILT to SILTY CLAY	5	4		.8	
11.100	36.42	16.17	.87	SANDY SILT to CLAYEY SILT	6	5		1.4	
11.250	36.91	10.39	1.06	CLAYEY SILT to SILTY CLAY	5	4		.8	
11.400	37.40	7.92	1.01	SENSITIVE FINE GRAINED	4	3		.6	
11.550	37.89	9.14	.66	SENSITIVE FINE GRAINED	5	4		.7	
11.700	38.39	15.83	.19	SANDY SILT to CLAYEY SILT	6	5		1.4	
11.850	38.88	23.07	.56	SILTY SAND to SANDY SILT	8	6	19		30.0
12.000	39.37	13.66	.88	SANDY SILT to CLAYEY SILT	5	4		1.1	
12.150	39.86	50.44	.75	SILTY SAND to SANDY SILT	17	14	41		35.5
12.300	40.35	142.91	.71	SAND	29	23	71		39.5
12.450	40.85	166.11	.69	SAND	33	27	75		40.5
12.600	41.34	164.48	.87	SAND	33	26	75		40.5
12.750	41.83	167.64	.87	SAND	34	27	75		40.5
12.900	42.32	177.37	.81	SAND	35	28	77		41.0
13.050	42.81	162.40	.84	SAND	32	26	74		40.0
13.200	43.31	130.85	.80	SAND to SILTY SAND	33	26	68		39.0
13.350	43.80	93.69	.67	SAND to SILTY SAND	23	18	58		38.0
13.500	44.29	56.36	.62	SAND to SILTY SAND	14	11	44		36.0
13.650	44.78	50.07	.84	SILTY SAND to SANDY SILT	17	13	40		34.5

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : CPT-02

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
13.800	45.28	81.01	.31	SAND to SILTY SAND	20	16	54		37.0
13.950	45.77	70.04	.73	SAND to SILTY SAND	18	14	49		36.5
14.100	46.26	75.72	.41	SAND to SILTY SAND	19	15	52		37.0
14.250	46.75	82.22	.61	SAND to SILTY SAND	21	16	54		37.0
14.400	47.24	80.33	.22	SAND to SILTY SAND	20	15	53		37.0
14.550	47.74	18.08	1.94	CLAYEY SILT to SILTY CLAY	9	7		1.2	
14.700	48.23	38.62	.60	SILTY SAND to SANDY SILT	13	10	32		32.0
14.850	48.72	40.73	.37	SILTY SAND to SANDY SILT	14	10	33		32.0
15.000	49.21	91.40	.90	SAND to SILTY SAND	23	17	56		37.5
15.150	49.70	111.37	.54	SAND	22	17	62		38.0
15.300	50.20	136.35	1.51	SAND to SILTY SAND	34	25	68		39.0
15.450	50.69	122.33	.76	SAND to SILTY SAND	31	23	64		38.5
15.600	51.18	43.87	1.50	SILTY SAND to SANDY SILT	15	11	35		32.5
15.750	51.67	33.27	3.22	CLAYEY SILT to SILTY CLAY	17	12		2.0	
15.900	52.17	53.83	2.25	SANDY SILT to CLAYEY SILT	22	16		3.4	
16.050	52.66	118.97	3.00	SANDY SILT to CLAYEY SILT	48	35		6.8	
16.200	53.15	167.05	2.76	SILTY SAND to SANDY SILT	56	41	73		39.5
16.350	53.64	65.80	2.39	SANDY SILT to CLAYEY SILT	26	19		4.2	
16.500	54.13	81.67	3.75	CLAYEY SILT to SILTY CLAY	41	30		4.6	
16.650	54.63	157.36	3.54	SANDY SILT to CLAYEY SILT	63	46		9.1	
16.800	55.12	210.26	2.10	SILTY SAND to SANDY SILT	70	51	79		40.5
16.950	55.61	64.20	3.15	SANDY SILT to CLAYEY SILT	26	18		3.6	
17.100	56.10	67.47	4.82	*VERY STIFF FINE GRAINED	67	48			
17.250	56.59	116.23	2.68	SILTY SAND to SANDY SILT	39	28	62		38.0
17.400	57.09	97.79	4.16	CLAYEY SILT to SILTY CLAY	49	35		5.6	
17.550	57.58	95.24	4.48	*VERY STIFF FINE GRAINED	95	68			
17.700	58.07	99.53	4.27	CLAYEY SILT to SILTY CLAY	50	35		5.7	
17.850	58.56	111.58	4.07	CLAYEY SILT to SILTY CLAY	56	39		6.4	
18.000	59.06	150.71	3.30	SANDY SILT to CLAYEY SILT	60	42		8.7	
18.150	59.55	181.64	3.15	SANDY SILT to CLAYEY SILT	73	51		10.5	
18.300	60.04	150.12	4.67	*VERY STIFF FINE GRAINED	100	100			
18.450	60.53	199.13	1.97	SAND to SILTY SAND	50	35	76		39.5
18.600	61.02	360.01	1.12	SAND	72	50	93		43.0
18.750	61.52	455.81	1.23	SAND	91	63	100		44.0
18.900	62.01	504.88	1.17	SAND	100	70	100		44.5
19.050	62.50	422.84	1.12	SAND	85	58	98		44.0
19.200	62.99	400.49	.94	SAND	80	55	96		43.5
19.350	63.48	454.13	.66	GRAVELLY SAND to SAND	76	52	99		44.0
19.500	63.98	264.94	3.17	*SAND to CLAYEY SAND	100	90			
19.650	64.47	150.99	4.13	*VERY STIFF FINE GRAINED	100	100			
19.800	64.96	87.51	3.71	CLAYEY SILT to SILTY CLAY	44	30		4.9	
19.950	65.45	206.56	1.20	SAND to SILTY SAND	52	35	77		39.5
20.100	65.94	268.83	1.04	SAND	54	36	84		41.5
20.250	66.44	262.71	1.18	SAND	53	35	83		41.0
20.400	66.93	265.67	1.15	SAND	53	36	83		41.5
20.550	67.42	279.37	1.72	SAND to SILTY SAND	70	47	85		41.5
20.700	67.91	91.65	4.41	*VERY STIFF FINE GRAINED	92	61			
20.850	68.41	64.86	4.52	CLAY to SILTY CLAY	43	29		3.6	
21.000	68.90	78.18	3.66	CLAYEY SILT to SILTY CLAY	39	26		4.4	
21.150	69.39	161.46	4.65	*VERY STIFF FINE GRAINED	100	100			

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : CPT-02

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
21.300	69.88	272.46	2.18	SILTY SAND to SANDY SILT	91	60	84		41.0
21.450	70.37	355.11	1.39	SAND	71	47	91		42.5
21.600	70.87	429.78	1.26	SAND	86	56	97		43.5
21.750	71.36	440.74	.98	SAND	88	58	97		43.5
21.900	71.85	419.82	.31	GRAVELLY SAND to SAND	70	46	96		43.0
22.050	72.34	243.40	1.77	SAND to SILTY SAND	61	40	80		40.0
22.200	72.83	234.42	1.19	SAND	47	30	79		40.0
22.350	73.33	259.63	1.45	SAND to SILTY SAND	65	42	82		40.5
22.500	73.82	192.33	2.49	SILTY SAND to SANDY SILT	64	41	73		39.0
22.650	74.31	258.68	1.81	SAND to SILTY SAND	65	42	82		40.5
22.800	74.80	281.92	1.49	SAND to SILTY SAND	70	45	84		41.0
22.950	75.30	351.92	*****		0	0			.0

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

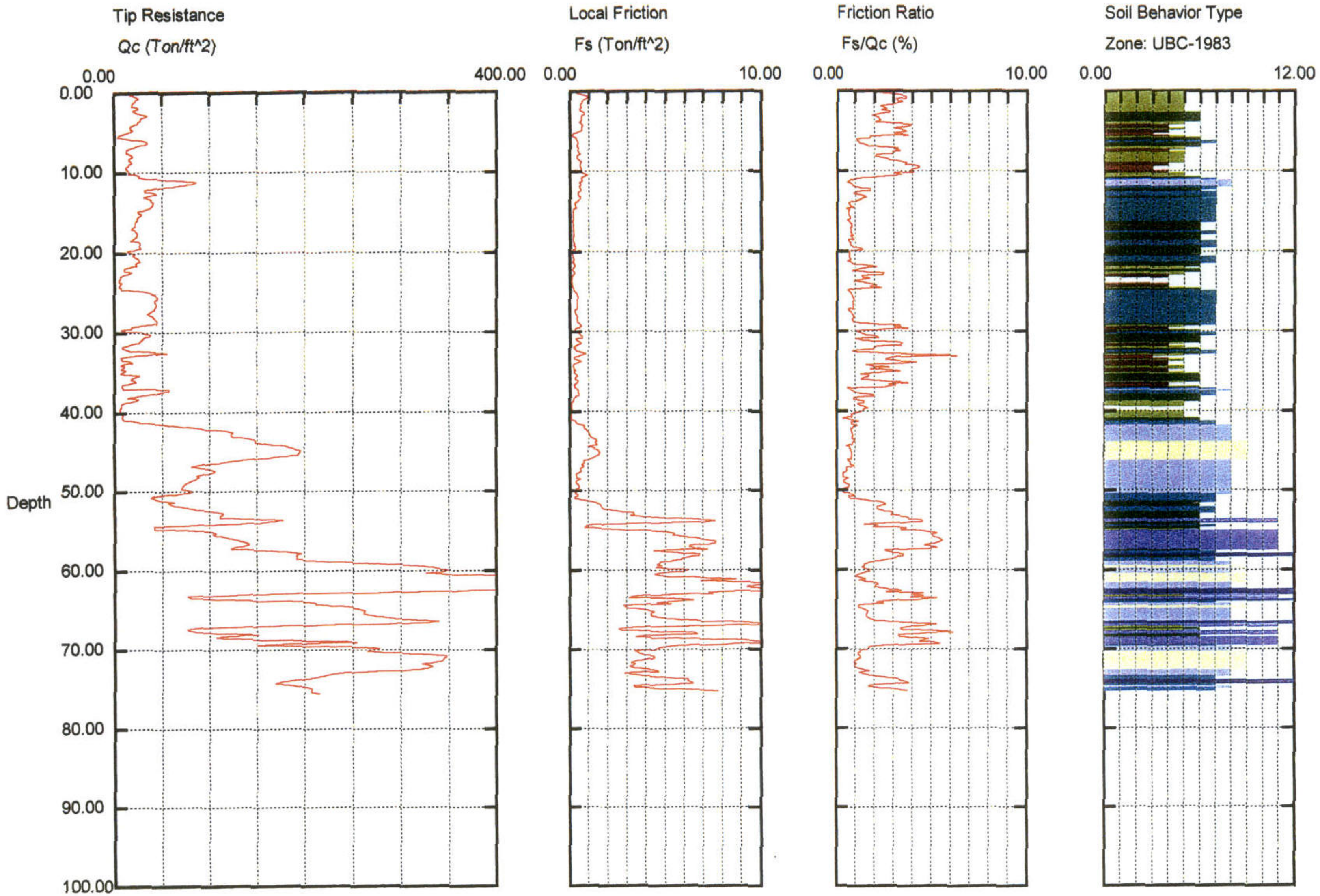
HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

Hushman Associates

Operator: ALAMEDA NAS #2
Sounding: SDF122
Cone Used: 408/GO-VO/R#4

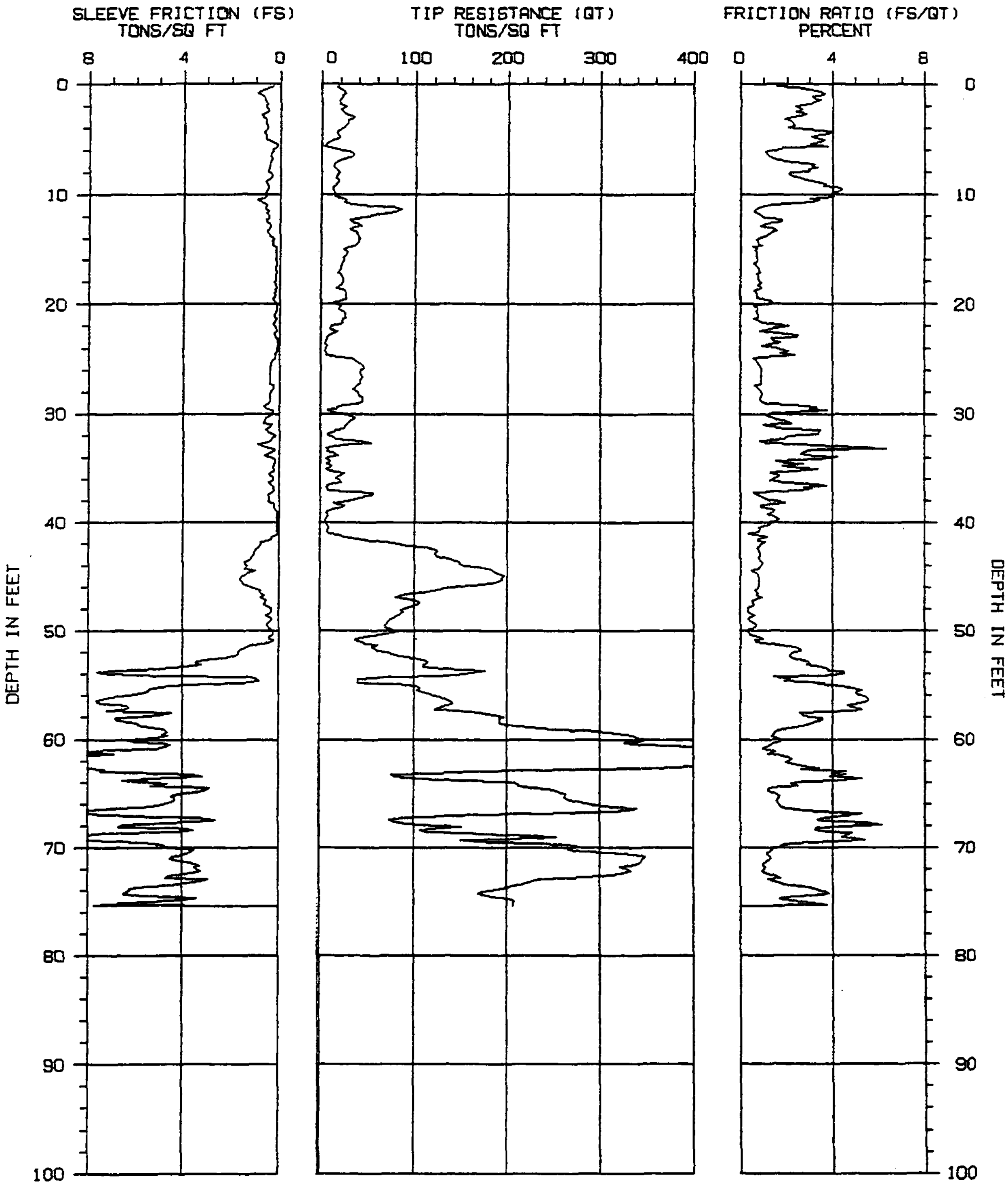
CPT Date/Time: 02-19-02 15:05
Location: CPT-03
Job Number: 010810



Maximum Depth = 75.62 feet

Depth Increment = 0.16 feet

1 sensitive fine grained
2 organic material
clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)



TIP RESISTANCE CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST

SOUNDING NUMBER: CPT-03

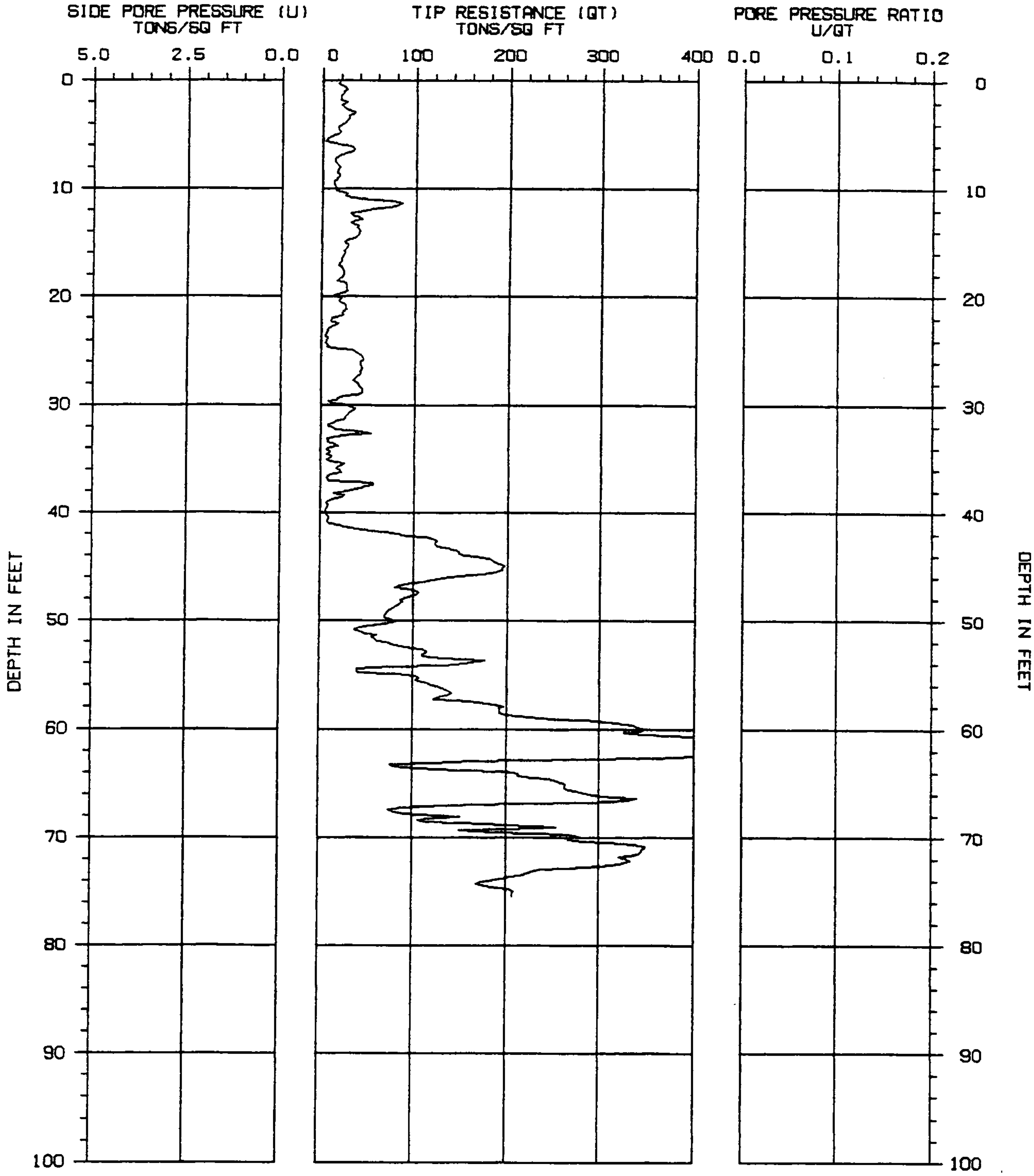
PROJECT NAME : ALAMEDA NAS #2

PROJECT NUMBER : 010810

CONE/RIG : 408/GO-VO/R#4

DATE/TIME: 02-19-02 15:05

HFA



TIP RESISTANCE CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST SOUNDING NUMBER: CPT-03

PROJECT NAME : ALAMEDA NAS #2 CONE/RIG : 408/GO-VO/R#4

PROJECT NUMBER : 010810 DATE/TIME: 02-19-02 15:05

HFA

CPT INTERPRETATIONS

SOUNDING : CPT-03
PROJECT : ALAMEDA NAS #2
DATE/TIME: 02-19-02 15:05

PROJECT No.: 010810
CONE/RIG : 408/GO-VO/R#4

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	20.65	3.00	CLAYEY SILT to SILTY CLAY	10	17		1.4	
.300	.98	22.69	3.61	CLAY to SILTY CLAY	15	24		1.5	
.450	1.48	19.38	3.46	CLAY to SILTY CLAY	13	21		1.3	
.600	1.97	25.56	2.35	CLAYEY SILT to SILTY CLAY	13	20		1.7	
.750	2.46	22.41	2.41	CLAYEY SILT to SILTY CLAY	11	18		1.5	
.900	2.95	34.57	2.23	SANDY SILT to CLAYEY SILT	14	22		2.3	
1.050	3.44	27.72	2.24	SANDY SILT to CLAYEY SILT	11	18		1.8	
1.200	3.94	24.60	2.07	SANDY SILT to CLAYEY SILT	10	16		1.9	
1.350	4.43	16.97	3.83	CLAY to SILTY CLAY	11	18		1.1	
1.500	4.92	17.21	3.49	CLAY to SILTY CLAY	11	18		1.1	
1.650	5.41	4.97	3.02	CLAY	5	8		.4	
1.800	5.91	17.23	1.74	CLAYEY SILT to SILTY CLAY	9	14		1.4	
1.950	6.40	34.91	1.17	SILTY SAND to SANDY SILT	12	19	46		39.5
2.100	6.89	21.07	1.66	SANDY SILT to CLAYEY SILT	8	13		1.7	
2.250	7.38	13.62	3.08	CLAY to SILTY CLAY	9	14		.9	
2.400	7.87	18.40	2.99	CLAYEY SILT to SILTY CLAY	9	14		1.2	
2.550	8.37	15.83	2.34	CLAYEY SILT to SILTY CLAY	8	11		1.2	
2.700	8.86	18.36	3.32	CLAY to SILTY CLAY	12	17		1.2	
2.850	9.35	13.07	4.13	CLAY	13	18		.8	
3.000	9.84	15.21	3.94	CLAY	15	20		1.0	
3.150	10.33	23.45	3.03	CLAYEY SILT to SILTY CLAY	12	15		1.5	
3.300	10.83	32.04	1.90	SANDY SILT to CLAYEY SILT	13	16		2.5	
3.450	11.32	85.79	.70	SAND to SILTY SAND	21	27	69		42.0
3.600	11.81	61.46	.88	SAND to SILTY SAND	15	19	59		39.5
3.750	12.30	31.31	1.79	SANDY SILT to CLAYEY SILT	13	15		2.4	
3.900	12.80	43.79	.91	SILTY SAND to SANDY SILT	15	17	48		38.0
4.050	13.29	34.50	1.57	SANDY SILT to CLAYEY SILT	14	16		2.7	
4.200	13.78	40.85	1.05	SILTY SAND to SANDY SILT	14	15	45		37.5
4.350	14.27	39.86	.73	SILTY SAND to SANDY SILT	13	15	43		37.0
4.500	14.76	30.74	.55	SILTY SAND to SANDY SILT	10	11	35		36.0
4.650	15.26	28.60	.66	SILTY SAND to SANDY SILT	10	10	33		35.5
4.800	15.75	23.73	.72	SILTY SAND to SANDY SILT	8	8	28		33.5
4.950	16.24	22.18	.59	SILTY SAND to SANDY SILT	7	8	25		32.5
5.100	16.73	21.65	.83	SANDY SILT to CLAYEY SILT	9	9		1.7	
5.250	17.22	21.44	.75	SANDY SILT to CLAYEY SILT	9	9		1.6	
5.400	17.72	23.86	.75	SILTY SAND to SANDY SILT	8	8	27		33.0
5.550	18.21	21.18	.76	SANDY SILT to CLAYEY SILT	8	9		1.6	
5.700	18.70	25.85	.62	SILTY SAND to SANDY SILT	9	9	29		33.5
5.850	19.19	26.75	.71	SILTY SAND to SANDY SILT	9	9	30		33.5
6.000	19.69	21.10	1.28	SANDY SILT to CLAYEY SILT	8	9		1.6	
6.150	20.18	21.73	.60	SILTY SAND to SANDY SILT	7	7	23		31.5

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : CPT-03

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
6.300	20.67	23.20	.69	SILTY SAND to SANDY SILT	8	8	25		32.0
6.450	21.16	26.94	.78	SILTY SAND to SANDY SILT	9	9	29		33.0
6.600	21.65	23.67	.97	SANDY SILT to CLAYEY SILT	9	9		1.8	
6.750	22.15	10.52	1.52	CLAYEY SILT to SILTY CLAY	5	5		.7	
6.900	22.64	14.77	1.62	CLAYEY SILT to SILTY CLAY	7	7		1.1	
7.050	23.13	6.65	1.35	SENSITIVE FINE GRAINED	3	3		.5	
7.200	23.62	4.82	1.24	SENSITIVE FINE GRAINED	2	2		.3	
7.350	24.11	5.61	1.78	CLAY to SILTY CLAY	4	4		.3	
7.500	24.61	6.80	2.35	CLAY to SILTY CLAY	5	4		.4	
7.650	25.10	36.86	.68	SILTY SAND to SANDY SILT	12	12	37		35.5
7.800	25.59	45.12	.82	SILTY SAND to SANDY SILT	15	14	42		36.5
7.950	26.08	42.89	.91	SILTY SAND to SANDY SILT	14	13	41		36.0
8.100	26.57	44.66	.90	SILTY SAND to SANDY SILT	15	14	42		36.0
8.250	27.07	42.32	.92	SILTY SAND to SANDY SILT	14	13	40		36.0
8.400	27.56	36.90	.73	SILTY SAND to SANDY SILT	12	11	36		34.5
8.550	28.05	39.94	.90	SILTY SAND to SANDY SILT	13	12	38		35.5
8.700	28.54	44.36	.86	SILTY SAND to SANDY SILT	15	13	41		36.0
8.850	29.04	39.86	1.23	SILTY SAND to SANDY SILT	13	12	38		35.0
9.000	29.53	17.14	3.03	CLAYEY SILT to SILTY CLAY	9	8		1.0	
9.150	30.02	29.06	1.07	SILTY SAND to SANDY SILT	10	9	28		32.0
9.300	30.51	36.37	1.57	SANDY SILT to CLAYEY SILT	15	13		2.8	
9.450	31.00	28.17	1.03	SILTY SAND to SANDY SILT	9	8	27		31.5
9.600	31.50	16.76	3.46	CLAY to SILTY CLAY	11	10		1.0	
9.750	31.99	9.20	2.07	CLAYEY SILT to SILTY CLAY	5	4		.6	
9.900	32.48	42.17	.88	SILTY SAND to SANDY SILT	14	12	38		35.0
10.050	32.97	16.87	4.15	CLAY	17	15		1.0	
10.200	33.46	8.69	2.76	CLAY to SILTY CLAY	6	5		.5	
10.350	33.96	10.64	4.23	CLAY	11	9		.6	
10.500	34.45	9.20	1.96	CLAYEY SILT to SILTY CLAY	5	4		.6	
10.650	34.94	10.62	2.64	CLAY to SILTY CLAY	7	6		.7	
10.800	35.43	26.32	1.29	SANDY SILT to CLAYEY SILT	11	9		1.9	
10.950	35.93	17.08	1.46	SANDY SILT to CLAYEY SILT	7	6		1.2	
11.100	36.42	14.94	2.74	CLAYEY SILT to SILTY CLAY	7	6		.9	
11.250	36.91	8.01	3.12	CLAY	8	7		.4	
11.400	37.40	56.55	.74	SAND to SILTY SAND	14	12	45		36.0
11.550	37.89	34.48	1.28	SILTY SAND to SANDY SILT	11	9	31		32.5
11.700	38.39	25.73	.89	SILTY SAND to SANDY SILT	9	7	22		30.5
11.850	38.88	10.24	1.46	CLAYEY SILT to SILTY CLAY	5	4		.6	
12.000	39.37	7.78	1.29	CLAYEY SILT to SILTY CLAY	4	3		.6	
12.150	39.86	6.10	1.48	SENSITIVE FINE GRAINED	3	2		.4	
12.300	40.35	8.94	.89	SENSITIVE FINE GRAINED	4	4		.7	
12.450	40.85	8.60	.81	SENSITIVE FINE GRAINED	4	3		.6	
12.600	41.34	26.47	1.17	SANDY SILT to CLAYEY SILT	11	8		1.9	
12.750	41.83	70.51	1.09	SILTY SAND to SANDY SILT	24	19	51		37.0
12.900	42.32	113.19	.76	SAND to SILTY SAND	28	23	64		38.5
13.050	42.81	122.18	.84	SAND to SILTY SAND	31	24	66		39.0
13.200	43.31	136.58	.83	SAND to SILTY SAND	34	27	69		39.0
13.350	43.80	147.76	.96	SAND to SILTY SAND	37	29	71		39.5
13.500	44.29	179.56	.80	SAND	36	28	77		40.5
13.650	44.78	191.54	.70	SAND	38	30	78		41.0

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : CPT-03

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
13.800	45.28	194.60	.83	SAND	39	30	79		41.0
13.950	45.77	163.84	.83	SAND	33	25	74		40.0
14.100	46.26	119.99	.62	SAND	24	18	65		38.5
14.250	46.75	86.40	.74	SAND to SILTY SAND	22	17	55		37.5
14.400	47.24	100.76	.55	SAND to SILTY SAND	25	19	59		38.0
14.550	47.74	99.62	.49	SAND to SILTY SAND	25	19	59		38.0
14.700	48.23	88.61	.37	SAND to SILTY SAND	22	17	56		37.5
14.850	48.72	79.99	.46	SAND to SILTY SAND	20	15	53		37.0
15.000	49.21	72.68	.54	SAND to SILTY SAND	18	14	50		36.5
15.150	49.70	69.90	.64	SAND to SILTY SAND	17	13	48		36.5
15.300	50.20	78.29	.36	SAND to SILTY SAND	20	15	52		36.5
15.450	50.69	40.00	.97	SILTY SAND to SANDY SILT	13	10	32		32.0
15.600	51.18	48.76	1.29	SILTY SAND to SANDY SILT	16	12	38		33.5
15.750	51.67	59.78	2.58	SANDY SILT to CLAYEY SILT	24	18		3.8	
15.900	52.17	78.37	2.17	SILTY SAND to SANDY SILT	26	19	51		36.5
16.050	52.66	109.39	2.52	SILTY SAND to SANDY SILT	36	27	61		38.0
16.200	53.15	110.64	2.89	SANDY SILT to CLAYEY SILT	44	32		6.3	
16.350	53.64	176.97	3.84	*SAND to CLAYEY SAND	88	65			
16.500	54.13	133.08	3.55	SANDY SILT to CLAYEY SILT	53	39		7.6	
16.650	54.63	41.36	1.93	SANDY SILT to CLAYEY SILT	17	12		2.5	
16.800	55.12	102.23	4.43	*VERY STIFF FINE GRAINED	100	74			
16.950	55.61	106.59	5.06	*VERY STIFF FINE GRAINED	100	77			
17.100	56.10	125.15	5.43	*VERY STIFF FINE GRAINED	100	90			
17.250	56.59	140.45	5.38	*VERY STIFF FINE GRAINED	100	100			
17.400	57.09	126.32	4.96	*VERY STIFF FINE GRAINED	100	90			
17.550	57.58	171.23	2.59	SILTY SAND to SANDY SILT	57	41	73		39.0
17.700	58.07	192.39	3.56	*SAND to CLAYEY SAND	96	68			
17.850	58.56	191.65	3.08	SILTY SAND to SANDY SILT	64	45	76		39.5
18.000	59.06	249.75	1.89	SAND to SILTY SAND	62	44	83		41.5
18.150	59.55	327.87	1.47	SAND to SILTY SAND	82	57	91		42.5
18.300	60.04	348.50	1.79	SAND to SILTY SAND	87	61	92		43.0
18.450	60.53	346.08	1.29	SAND	69	48	92		43.0
18.600	61.02	560.42	1.27	SAND	100	78	100		
18.750	61.52	535.52	1.59	SAND	100	74	100		
18.900	62.01	464.69	2.07	SAND to SILTY SAND	100	80	100		44.0
19.050	62.50	443.36	2.73	*SAND to CLAYEY SAND	100	100			
19.200	62.99	163.29	4.56	*VERY STIFF FINE GRAINED	100	100			
19.350	63.48	80.43	3.89	CLAYEY SILT to SILTY CLAY	40	28		4.5	
19.500	63.98	199.55	2.83	SILTY SAND to SANDY SILT	67	45	76		39.5
19.650	64.47	226.68	1.26	SAND to SILTY SAND	57	39	79		40.0
19.800	64.96	258.42	1.43	SAND to SILTY SAND	65	44	83		41.0
19.950	65.45	261.78	1.65	SAND to SILTY SAND	65	44	83		41.5
20.100	65.94	284.45	1.62	SAND to SILTY SAND	71	48	86		42.0
20.250	66.44	339.13	2.04	SAND to SILTY SAND	85	57	91		42.5
20.400	66.93	212.26	5.27	*VERY STIFF FINE GRAINED	100	100			
20.550	67.42	75.82	3.39	SANDY SILT to CLAYEY SILT	30	20		4.2	
20.700	67.91	106.12	6.14	*VERY STIFF FINE GRAINED	100	71			
20.850	68.41	107.16	3.27	SANDY SILT to CLAYEY SILT	43	28		6.1	
21.000	68.90	200.10	4.62	*VERY STIFF FINE GRAINED	100	100			
21.150	69.39	150.60	5.44	*VERY STIFF FINE GRAINED	100	99			

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

SOUNDING : CPT-03

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
21.300	69.88	277.69	1.68	SAND to SILTY SAND	69	46	84		41.5
21.450	70.37	281.94	1.29	SAND	56	37	85		41.5
21.600	70.87	348.03	1.26	SAND	70	46	91		42.5
21.750	71.36	344.14	1.12	SAND	69	45	90		42.5
21.900	71.85	322.24	1.07	SAND	64	42	88		42.0
22.050	72.34	328.30	1.03	SAND	66	43	89		42.0
22.200	72.83	265.33	1.76	SAND to SILTY SAND	66	43	82		40.5
22.350	73.33	220.58	1.73	SAND to SILTY SAND	55	36	77		39.5
22.500	73.82	196.26	3.17	SANDY SILT to CLAYEY SILT	79	51		11.3	
22.650	74.31	169.07	3.83	*SAND to CLAYEY SAND	85	54			
22.800	74.80	202.38	1.70	SAND to SILTY SAND	51	32	74		39.0
22.950	75.30	206.41	3.74	*SAND to CLAYEY SAND	100	66			

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

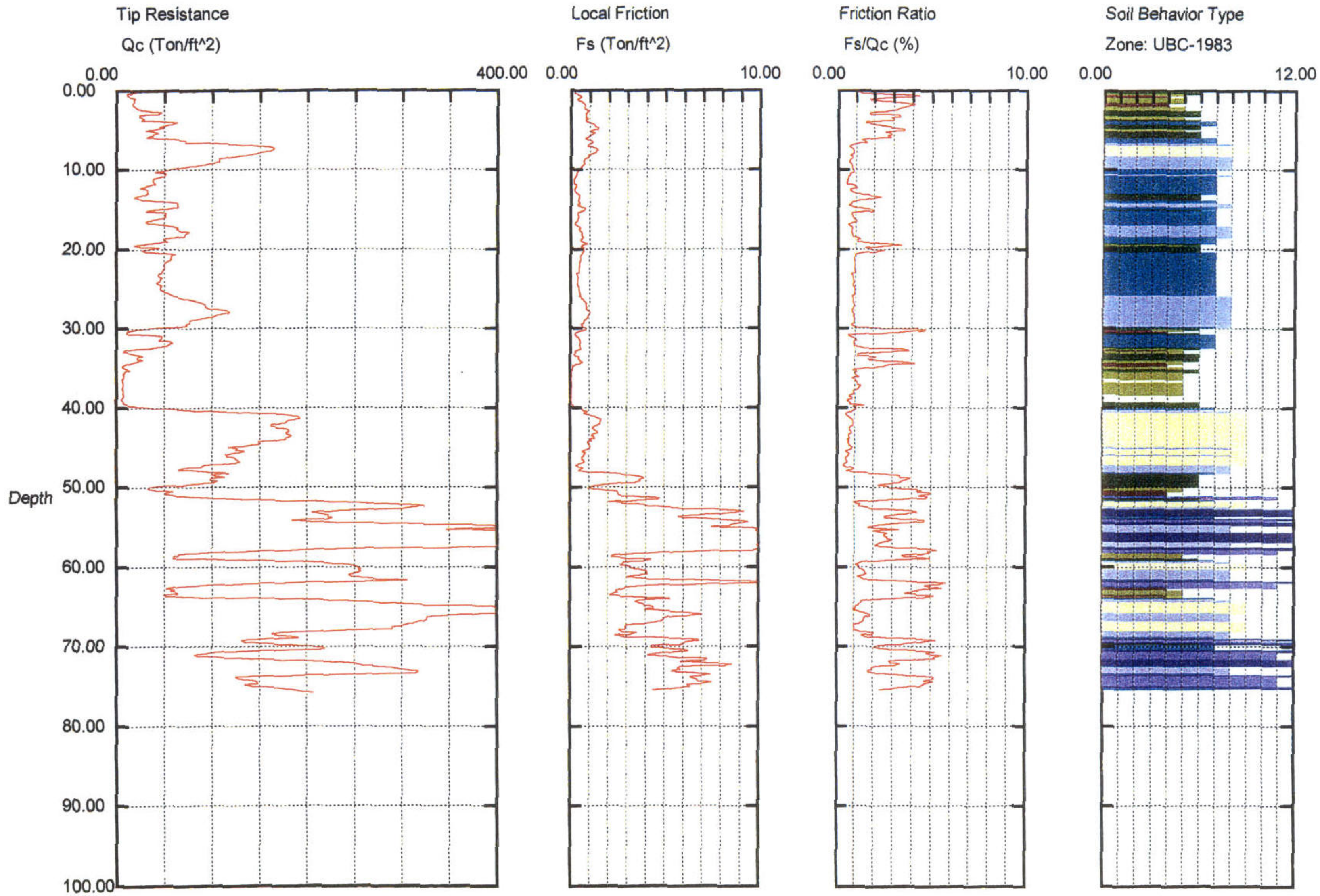
HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

Hushmand Associates

Operator: ALAMEDA NAS #2
Sounding: SDF123
Cone Used: 408/GO-VO/R#4

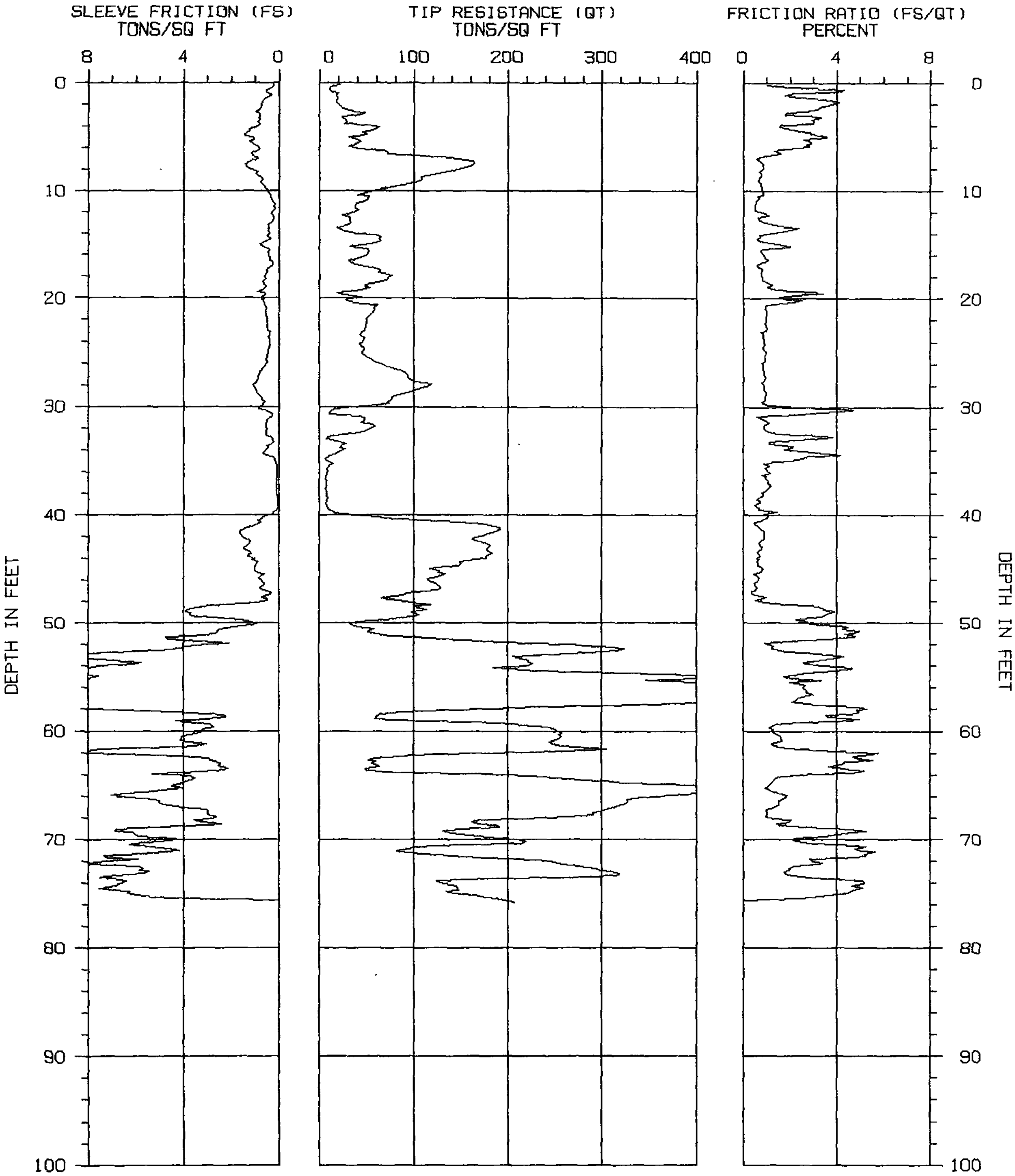
CPT Date/Time: 02-20-02 07:45
Location: CPT-04
Job Number: 010810



Maximum Depth = 75.79 feet

Depth Increment = 0.16 feet

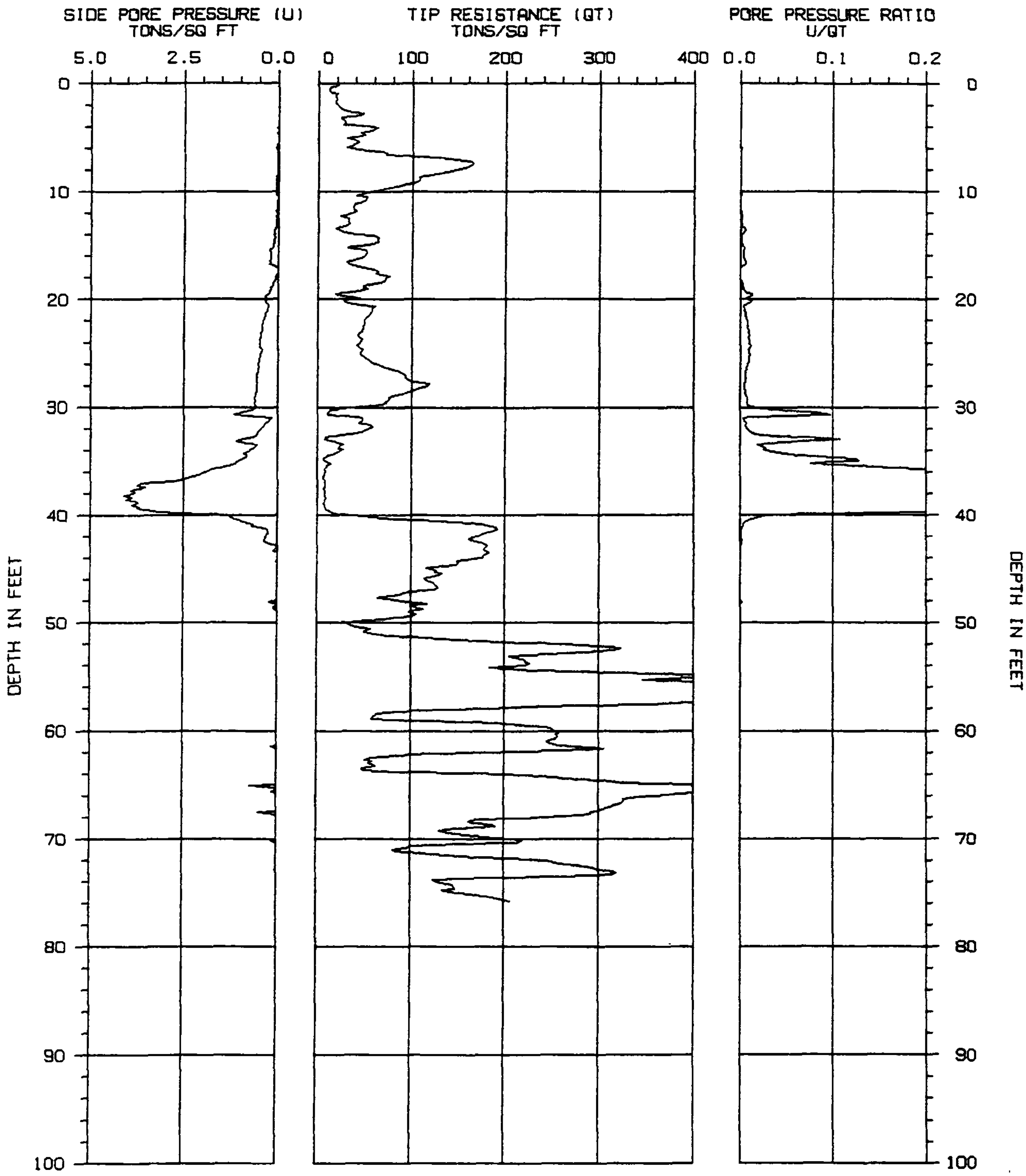
sensitive fine grained
organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)



TIP RESISTANCE CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST	SOUNDING NUMBER: CPT-04
PROJECT NAME : ALAMEDA NAS #2	CONE/RIG : 408/GO-VO/R#4
PROJECT NUMBER : 010810	DATE/TIME: 02-20-02 07:45

HFA



TIP RESISTANCE CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST — SOUNDING NUMBER: CPT-04

PROJECT NAME : ALAMEDA NAS #2 — CONE/RIG : 408/GO-VD/R#4

PROJECT NUMBER : 010810 — DATE/TIME: 02-20-02 07:45

HFA

CPT INTERPRETATIONS

SOUNDING : CPT-04 PROJECT No.: 010810
PROJECT : ALAMEDA NAS #2 CONE/RIG : 408/GO-VO/R#4
DATE/TIME: 02-20-02 07:45

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	11.51	2.87	CLAY to SILTY CLAY	8	12		.8	
.300	.98	20.29	2.17	CLAYEY SILT to SILTY CLAY	10	16		1.6	
.450	1.48	19.08	3.04	CLAYEY SILT to SILTY CLAY	10	15		1.3	
.600	1.97	19.95	3.76	CLAY to SILTY CLAY	13	21		1.3	
.750	2.46	27.02	3.18	CLAYEY SILT to SILTY CLAY	14	22		1.8	
.900	2.95	45.08	1.77	SILTY SAND to SANDY SILT	15	24	53		45.0
1.050	3.44	27.66	2.93	CLAYEY SILT to SILTY CLAY	14	22		1.8	
1.200	3.94	52.07	1.63	SILTY SAND to SANDY SILT	17	28	58		44.5
1.350	4.43	52.22	2.41	SANDY SILT to CLAYEY SILT	21	33		3.5	
1.500	4.92	40.17	3.56	CLAYEY SILT to SILTY CLAY	20	32		2.7	
1.650	5.41	43.87	2.85	SANDY SILT to CLAYEY SILT	18	28		2.9	
1.800	5.91	31.87	2.82	CLAYEY SILT to SILTY CLAY	16	25		2.1	
1.950	6.40	72.49	1.49	SILTY SAND to SANDY SILT	24	39	67		43.5
2.100	6.89	129.27	.71	SAND	26	41	84		46.0
2.250	7.38	164.65	.80	SAND	33	51	91		46.5
2.400	7.87	150.58	.86	SAND	30	45	88		46.0
2.550	8.37	125.34	.78	SAND to SILTY SAND	31	45	83		45.0
2.700	8.86	108.20	.67	SAND to SILTY SAND	27	38	79		44.0
2.850	9.35	94.16	.76	SAND to SILTY SAND	24	32	74		43.0
3.000	9.84	62.69	.80	SAND to SILTY SAND	16	21	62		40.5
3.150	10.33	41.11	.88	SILTY SAND to SANDY SILT	14	18	49		38.5
3.300	10.83	50.27	.58	SAND to SILTY SAND	13	16	54		39.0
3.450	11.32	38.50	.49	SILTY SAND to SANDY SILT	13	16	46		38.0
3.600	11.81	41.21	.51	SILTY SAND to SANDY SILT	14	17	47		38.0
3.750	12.30	25.24	1.07	SANDY SILT to CLAYEY SILT	10	12		2.0	
3.900	12.80	32.65	.80	SILTY SAND to SANDY SILT	11	13	39		37.0
4.050	13.29	25.20	1.83	SANDY SILT to CLAYEY SILT	10	12		2.0	
4.200	13.78	27.04	1.59	SANDY SILT to CLAYEY SILT	11	12		2.1	
4.350	14.27	64.44	.74	SAND to SILTY SAND	16	18	57		39.0
4.500	14.76	64.03	.95	SAND to SILTY SAND	16	17	57		38.5
4.650	15.26	32.33	1.98	SANDY SILT to CLAYEY SILT	13	14		2.1	
4.800	15.75	51.56	.83	SILTY SAND to SANDY SILT	17	18	50		38.0
4.950	16.24	42.36	.94	SILTY SAND to SANDY SILT	14	15	44		37.0
5.100	16.73	33.91	.74	SILTY SAND to SANDY SILT	11	12	37		36.0
5.250	17.22	60.55	.79	SAND to SILTY SAND	15	16	54		38.5
5.400	17.72	66.92	.76	SAND to SILTY SAND	17	17	56		38.5
5.550	18.21	71.62	.81	SAND to SILTY SAND	18	18	58		38.5
5.700	18.70	49.22	1.26	SILTY SAND to SANDY SILT	16	17	47		37.5
5.850	19.19	45.06	1.46	SILTY SAND to SANDY SILT	15	15	45		37.0
6.000	19.69	25.62	2.22	SANDY SILT to CLAYEY SILT	10	10		1.6	
6.150	20.18	27.79	2.52	CLAYEY SILT to SILTY CLAY	14	14		1.8	

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : CPT-04

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
6.300	20.67	61.84	.97	SILTY SAND to SANDY SILT	21	20	53		38.0
6.450	21.16	56.79	.99	SILTY SAND to SANDY SILT	19	19	50		38.0
6.600	21.65	52.01	1.00	SILTY SAND to SANDY SILT	17	17	48		37.5
6.750	22.15	50.05	1.00	SILTY SAND to SANDY SILT	17	16	46		37.0
6.900	22.64	49.18	.98	SILTY SAND to SANDY SILT	16	16	46		37.0
7.050	23.13	44.85	.78	SILTY SAND to SANDY SILT	15	14	43		36.5
7.200	23.62	48.12	.87	SILTY SAND to SANDY SILT	16	15	45		37.0
7.350	24.11	45.23	.91	SILTY SAND to SANDY SILT	15	14	43		36.5
7.500	24.61	47.95	.86	SILTY SAND to SANDY SILT	16	15	44		36.5
7.650	25.10	46.27	.97	SILTY SAND to SANDY SILT	15	15	43		36.5
7.800	25.59	54.96	.93	SILTY SAND to SANDY SILT	18	17	48		37.0
7.950	26.08	64.63	.85	SAND to SILTY SAND	16	15	52		38.0
8.100	26.57	83.51	.86	SAND to SILTY SAND	21	19	60		38.5
8.250	27.07	93.58	.91	SAND to SILTY SAND	23	22	63		39.0
8.400	27.56	97.64	.89	SAND to SILTY SAND	24	22	64		39.0
8.550	28.05	117.08	.91	SAND to SILTY SAND	29	27	69		39.5
8.700	28.54	100.70	.93	SAND to SILTY SAND	25	23	64		39.0
8.850	29.04	76.97	.96	SAND to SILTY SAND	19	17	56		38.0
9.000	29.53	72.78	.80	SAND to SILTY SAND	18	16	55		38.0
9.150	30.02	35.35	2.35	SANDY SILT to CLAYEY SILT	14	13		2.2	
9.300	30.51	11.39	3.34	CLAY to SILTY CLAY	8	7		.6	
9.450	31.00	47.82	.73	SILTY SAND to SANDY SILT	16	14	42		36.0
9.600	31.50	44.91	1.11	SILTY SAND to SANDY SILT	15	13	40		36.0
9.750	31.99	55.24	.92	SILTY SAND to SANDY SILT	18	16	46		36.5
9.900	32.48	36.22	1.49	SILTY SAND to SANDY SILT	12	10	34		33.5
10.050	32.97	8.48	2.95	CLAY to SILTY CLAY	6	5		.4	
10.200	33.46	28.36	1.13	SILTY SAND to SANDY SILT	9	8	27		31.5
10.350	33.96	28.04	1.78	SANDY SILT to CLAYEY SILT	11	10		2.1	
10.500	34.45	12.96	4.17	CLAY	13	11		.7	
10.650	34.94	7.75	2.06	CLAY to SILTY CLAY	5	4		.5	
10.800	35.43	9.95	1.11	CLAYEY SILT to SILTY CLAY	5	4		.8	
10.950	35.93	8.78	1.03	CLAYEY SILT to SILTY CLAY	4	4		.7	
11.100	36.42	7.13	.98	SENSITIVE FINE GRAINED	4	3		.5	
11.250	36.91	8.35	1.08	CLAYEY SILT to SILTY CLAY	4	3		.6	
11.400	37.40	8.05	1.12	SENSITIVE FINE GRAINED	4	3		.6	
11.550	37.89	8.45	.83	SENSITIVE FINE GRAINED	4	3		.6	
11.700	38.39	8.84	.68	SENSITIVE FINE GRAINED	4	4		.7	
11.850	38.88	6.92	.72	SENSITIVE FINE GRAINED	3	3		.5	
12.000	39.37	8.81	.68	SENSITIVE FINE GRAINED	4	4		.7	
12.150	39.86	22.80	.92	SANDY SILT to CLAYEY SILT	9	7		1.6	
12.300	40.35	81.45	.90	SAND to SILTY SAND	20	16	55		37.5
12.450	40.85	175.99	.52	SAND	35	28	77		41.0
12.600	41.34	192.50	.79	SAND	39	31	79		41.5
12.750	41.83	176.71	.91	SAND	35	28	77		41.0
12.900	42.32	161.95	.80	SAND	32	26	74		40.0
13.050	42.81	180.86	.74	SAND	36	29	77		41.0
13.200	43.31	178.88	.69	SAND	36	28	77		40.5
13.350	43.80	176.93	.72	SAND	35	28	76		40.5
13.500	44.29	149.01	.60	SAND	30	23	71		39.5
13.650	44.78	133.54	.78	SAND	27	21	68		39.0

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : CPT-04

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
13.800	45.28	125.43	.69	SAND	25	19	66		38.5
13.950	45.77	119.16	.70	SAND to SILTY SAND	30	23	65		38.5
14.100	46.26	123.50	.51	SAND	25	19	66		38.5
14.250	46.75	129.06	.60	SAND	26	20	67		39.0
14.400	47.24	92.01	.36	SAND to SILTY SAND	23	18	57		37.5
14.550	47.74	65.31	.84	SAND to SILTY SAND	16	12	47		36.0
14.700	48.23	117.44	1.82	SILTY SAND to SANDY SILT	39	30	64		38.5
14.850	48.72	113.06	3.41	SANDY SILT to CLAYEY SILT	45	34		6.5	
15.000	49.21	105.46	3.52	SANDY SILT to CLAYEY SILT	42	32		6.0	
15.150	49.70	69.83	2.26	SILTY SAND to SANDY SILT	23	17	48		36.5
15.300	50.20	34.46	3.48	CLAYEY SILT to SILTY CLAY	17	13		2.1	
15.450	50.69	51.14	4.99	CLAY to SILTY CLAY	34	25		2.8	
15.600	51.18	76.23	4.81	*VERY STIFF FINE GRAINED	76	57			
15.750	51.67	171.93	1.90	SILTY SAND to SANDY SILT	57	42	74		39.5
15.900	52.17	297.64	1.32	SAND	60	44	89		42.5
16.050	52.66	303.01	1.97	SAND to SILTY SAND	76	56	90		43.0
16.200	53.15	205.86	4.29	*VERY STIFF FINE GRAINED	100	100			
16.350	53.64	222.48	2.59	SILTY SAND to SANDY SILT	74	54	81		41.0
16.500	54.13	184.04	4.68	*VERY STIFF FINE GRAINED	100	100			
16.650	54.63	323.24	2.66	SILTY SAND to SANDY SILT	100	78	91		43.0
16.800	55.12	460.57	2.11	SAND to SILTY SAND	100	83	100		44.5
16.950	55.61	517.20	2.38	*SAND to CLAYEY SAND	100	100			
17.100	56.10	498.15	2.59	*SAND to CLAYEY SAND	100	100			
17.250	56.59	433.52	2.96	*SAND to CLAYEY SAND	100	100			
17.400	57.09	468.43	2.28	SAND to SILTY SAND	100	83	100		44.5
17.550	57.58	351.62	3.13	*SAND to CLAYEY SAND	100	100			
17.700	58.07	136.77	4.85	*VERY STIFF FINE GRAINED	100	97			
17.850	58.56	63.90	3.55	CLAYEY SILT to SILTY CLAY	32	23		3.6	
18.000	59.06	105.86	4.06	CLAYEY SILT to SILTY CLAY	53	37		6.0	
18.150	59.55	226.87	1.23	SAND	45	32	80		40.5
18.300	60.04	253.56	1.26	SAND	51	35	83		41.5
18.450	60.53	256.91	1.59	SAND to SILTY SAND	64	45	84		41.5
18.600	61.02	244.36	1.52	SAND to SILTY SAND	61	42	82		41.0
18.750	61.52	273.04	1.52	SAND to SILTY SAND	68	47	85		42.0
18.900	62.01	176.88	5.79	*VERY STIFF FINE GRAINED	100	100			
19.050	62.50	59.38	4.95	CLAY to SILTY CLAY	40	27		3.3	
19.200	62.99	56.34	4.33	CLAYEY SILT to SILTY CLAY	28	19		3.1	
19.350	63.48	49.18	4.31	CLAY to SILTY CLAY	33	22		2.7	
19.500	63.98	170.36	3.12	SANDY SILT to CLAYEY SILT	68	46		9.8	
19.650	64.47	277.33	1.33	SAND	55	38	85		42.0
19.800	64.96	396.87	1.11	SAND	79	54	95		43.5
19.950	65.45	421.86	1.17	SAND	84	57	97		43.5
20.100	65.94	375.37	1.87	SAND to SILTY SAND	94	63	94		43.0
20.250	66.44	326.89	1.53	SAND to SILTY SAND	82	55	90		42.5
20.400	66.93	316.85	1.44	SAND	63	42	89		42.0
20.550	67.42	299.42	.99	SAND	60	40	87		42.0
20.700	67.91	265.58	.98	SAND	53	35	83		41.0
20.850	68.41	162.80	1.97	SILTY SAND to SANDY SILT	54	36	69		38.5
21.000	68.90	190.67	2.66	SILTY SAND to SANDY SILT	64	42	74		39.0
21.150	69.39	134.10	4.57	*VERY STIFF FINE GRAINED	100	89			

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : CPT-04

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
21.300	69.88	189.95	2.28	SILTY SAND to SANDY SILT	63	42	73		39.0
21.450	70.37	214.68	2.84	SILTY SAND to SANDY SILT	72	47	77		39.5
21.600	70.87	95.73	4.82	*VERY STIFF FINE GRAINED	96	63			
21.750	71.36	110.47	5.21	*VERY STIFF FINE GRAINED	100	72			
21.900	71.85	207.37	2.85	SILTY SAND to SANDY SILT	69	45	76		39.5
22.050	72.34	258.42	3.19	*SAND to CLAYEY SAND	100	84			
22.200	72.83	298.06	1.97	SAND to SILTY SAND	75	48	86		41.5
22.350	73.33	315.17	2.16	SAND to SILTY SAND	79	51	87		42.0
22.500	73.82	124.98	5.12	*VERY STIFF FINE GRAINED	100	81			
22.650	74.31	145.40	4.68	*VERY STIFF FINE GRAINED	100	93			
22.800	74.80	134.76	4.62	*VERY STIFF FINE GRAINED	100	86			
22.950	75.30	177.61	2.95	SILTY SAND to SANDY SILT	59	38	71		38.5
23.100	75.79	206.46	*****		0	0			.0

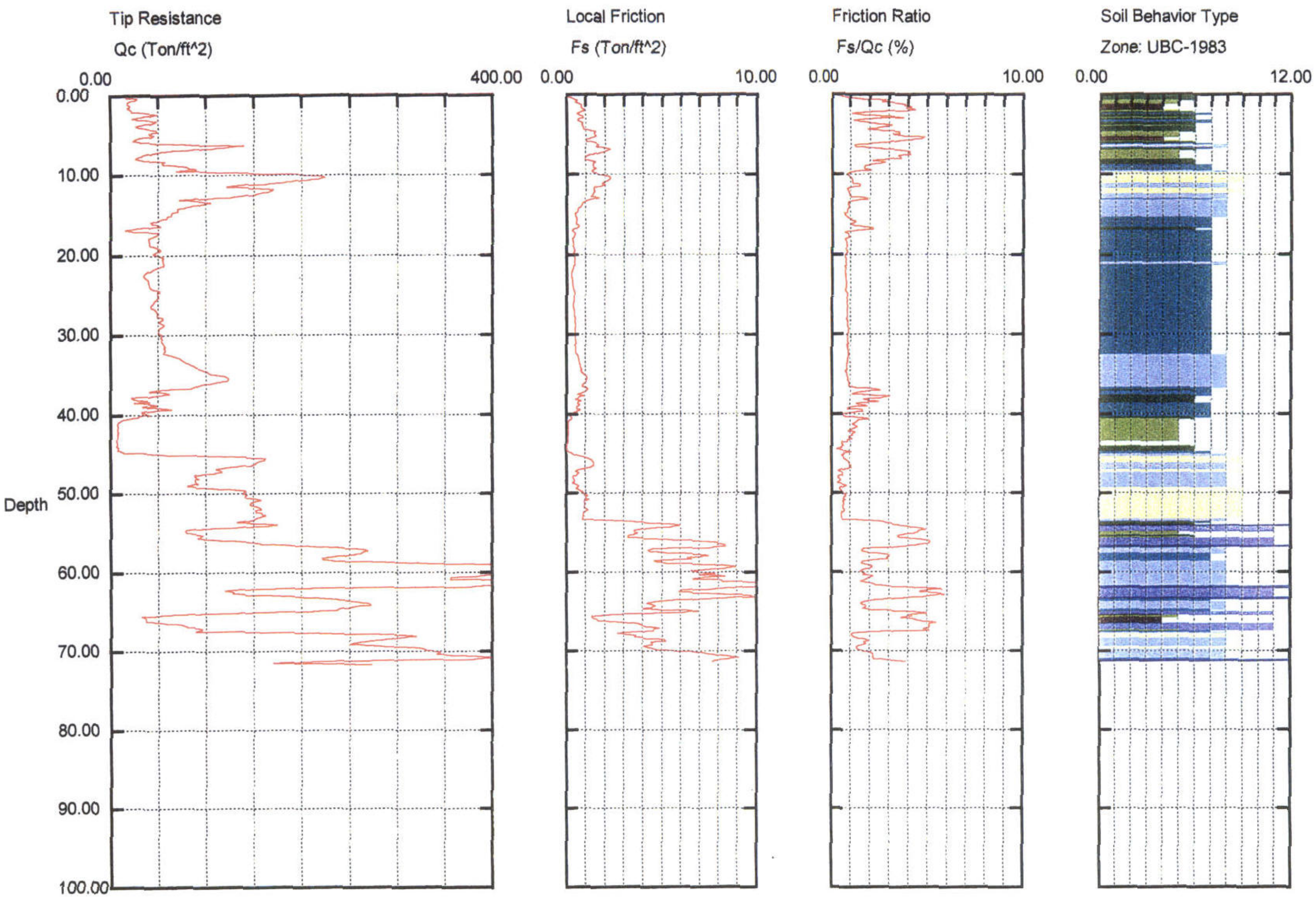
TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

Interpretations based on: Robertson and Campanella, 1989.

Hushman[illegible] Associates

Operator: ALAMEDA NAS #2
Sounding: SDF124
Cone Used: 408/GO-VO/R#4

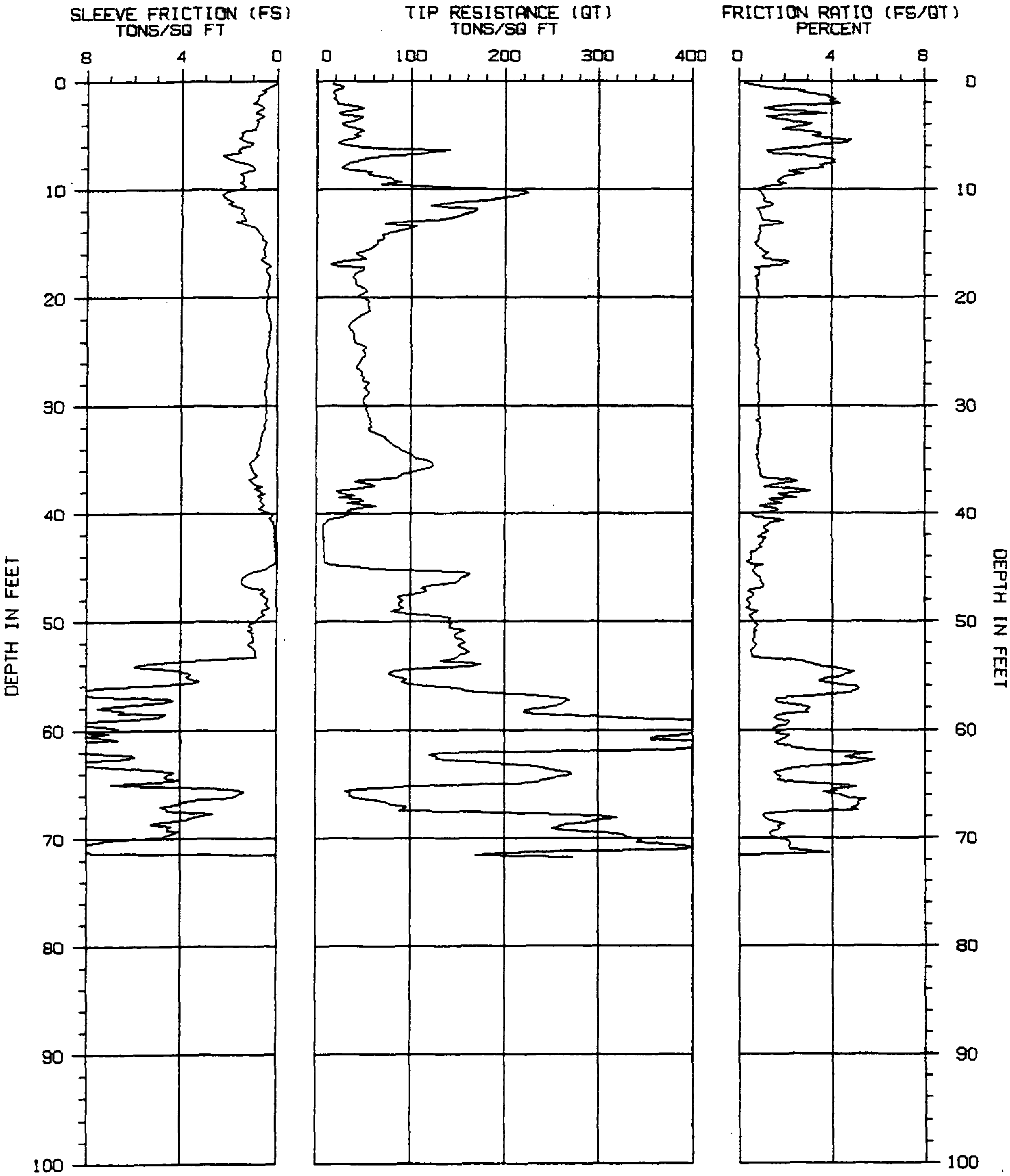
CPT Date/Time: 02-20-02 09:52
Location: CPT-05
Job Number: 010810



Maximum Depth = 71.69 feet

Depth Increment = 0.16 feet

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)



TIP RESISTANCE CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST

SOUNDING NUMBER: CPT-05

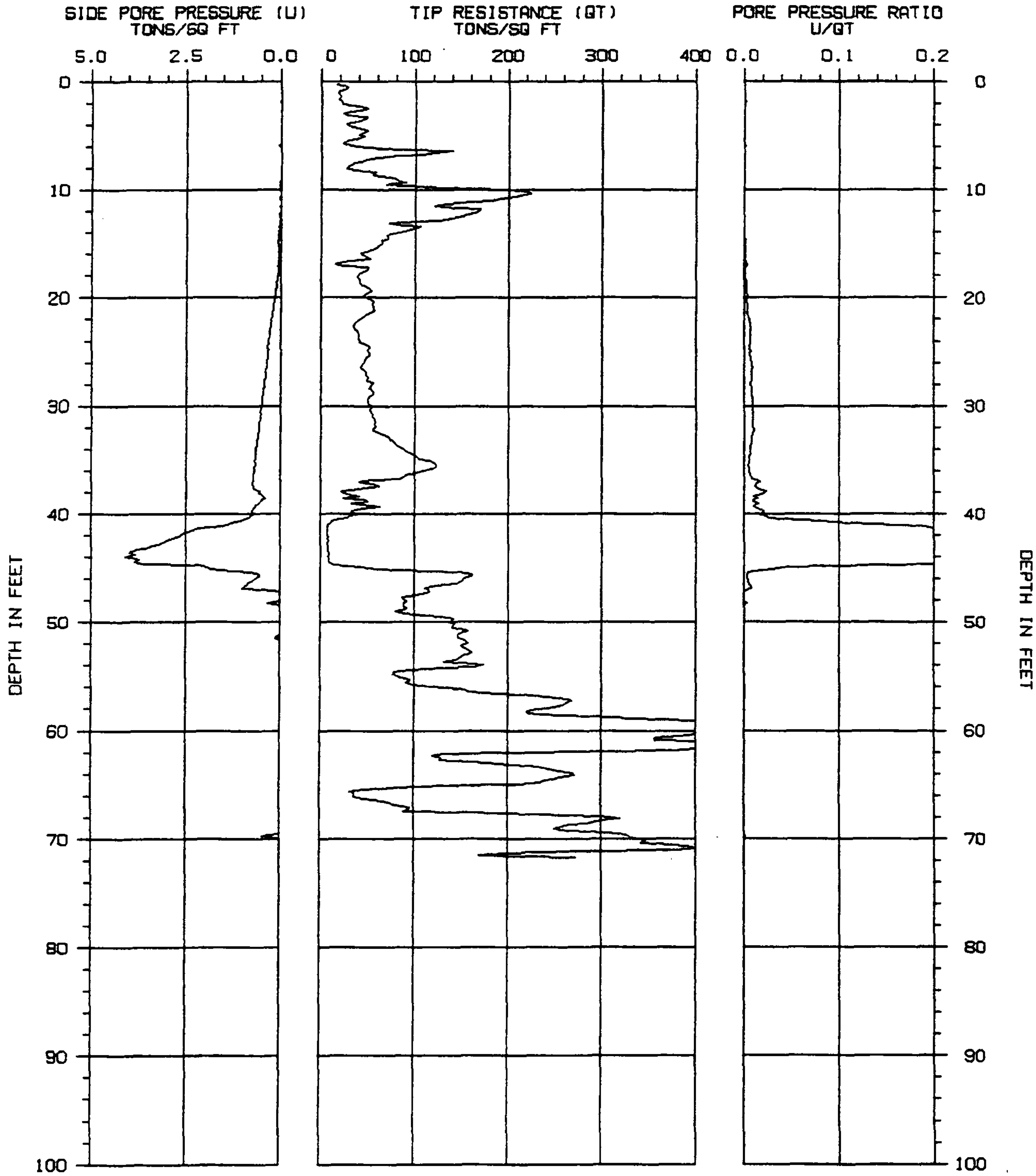
PROJECT NAME : ALAMEDA NAS #2

PROJECT NUMBER : 010810

CONE/RIG : 408/GO-VO/R#4

DATE/TIME: 02-20-02 09:52

HFA



TIP RESISTANCE CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST SOUNDING NUMBER: CPT-05

PROJECT NAME : ALAMEDA NAS #2	CONE/RIG : 408/GO-VO/R#4
PROJECT NUMBER : 010810	DATE/TIME: 02-20-02 09:52

HFA

CPT INTERPRETATIONS

SOUNDING : CPT-05
PROJECT : ALAMEDA NAS #2
DATE/TIME: 02-20-02 09:52

PROJECT No.: 010810
CONE/RIG : 408/GO-VO/R#4

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	28.66	.97	SILTY SAND to SANDY SILT	10	15	41		
.300	.98	18.23	2.68	CLAYEY SILT to SILTY CLAY	9	15		1.2	
.450	1.48	20.93	3.84	CLAY to SILTY CLAY	14	22		1.4	
.600	1.97	22.90	4.38	CLAY to SILTY CLAY	15	24		1.5	
.750	2.46	49.05	1.14	SILTY SAND to SANDY SILT	16	26	56		46.0
.900	2.95	23.62	3.76	CLAY to SILTY CLAY	16	25		1.6	
1.050	3.44	46.70	1.52	SILTY SAND to SANDY SILT	16	25	55		44.5
1.200	3.94	27.13	3.17	CLAYEY SILT to SILTY CLAY	14	22		1.8	
1.350	4.43	47.65	1.91	SANDY SILT to CLAYEY SILT	19	30		3.2	
1.500	4.92	40.66	3.56	CLAYEY SILT to SILTY CLAY	20	33		2.4	
1.650	5.41	30.49	4.84	CLAY	30	49		1.8	
1.800	5.91	31.36	3.36	CLAYEY SILT to SILTY CLAY	16	25		2.1	
1.950	6.40	139.83	1.20	SAND to SILTY SAND	35	56	86		46.5
2.100	6.89	82.64	2.78	SANDY SILT to CLAYEY SILT	33	53		4.8	
2.250	7.38	42.83	4.01	CLAYEY SILT to SILTY CLAY	21	33		2.5	
2.400	7.87	28.43	3.61	CLAYEY SILT to SILTY CLAY	14	21		1.9	
2.550	8.37	57.74	2.19	SANDY SILT to CLAYEY SILT	23	33		3.8	
2.700	8.86	71.19	1.97	SILTY SAND to SANDY SILT	24	33	67		42.0
2.850	9.35	90.21	1.74	SILTY SAND to SANDY SILT	30	41	73		43.0
3.000	9.84	157.17	.93	SAND	31	42	88		45.0
3.150	10.33	224.15	1.00	SAND	45	58	98		46.0
3.300	10.83	190.61	1.12	SAND	38	48	92		45.5
3.450	11.32	133.20	1.51	SAND to SILTY SAND	33	41	81		44.0
3.600	11.81	170.04	.84	SAND	34	41	88		44.5
3.750	12.30	156.34	.94	SAND	31	37	85		44.0
3.900	12.80	127.07	1.05	SAND to SILTY SAND	32	37	78		43.0
4.050	13.29	88.51	1.34	SILTY SAND to SANDY SILT	30	34	67		41.0
4.200	13.78	92.29	.90	SAND to SILTY SAND	23	26	68		41.0
4.350	14.27	70.51	.92	SAND to SILTY SAND	18	19	60		39.0
4.500	14.76	64.65	.85	SAND to SILTY SAND	16	18	57		39.0
4.650	15.26	59.80	.82	SAND to SILTY SAND	15	16	54		38.5
4.800	15.75	48.99	1.11	SILTY SAND to SANDY SILT	16	17	48		38.0
4.950	16.24	49.63	1.02	SILTY SAND to SANDY SILT	17	17	48		38.0
5.100	16.73	24.13	2.17	SANDY SILT to CLAYEY SILT	10	10		1.5	
5.250	17.22	50.24	.73	SILTY SAND to SANDY SILT	17	17	48		38.0
5.400	17.72	44.15	.91	SILTY SAND to SANDY SILT	15	15	45		37.0
5.550	18.21	40.05	.77	SILTY SAND to SANDY SILT	13	14	42		36.5
5.700	18.70	41.19	.81	SILTY SAND to SANDY SILT	14	14	42		36.5
5.850	19.19	51.09	.79	SILTY SAND to SANDY SILT	17	17	48		37.5
6.000	19.69	47.44	.88	SILTY SAND to SANDY SILT	16	16	46		37.0
6.150	20.18	52.39	.76	SILTY SAND to SANDY SILT	17	17	48		37.5

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : CPT-05

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
6.300	20.67	55.09	.80	SILTY SAND to SANDY SILT	18	18	50		38.0
6.450	21.16	56.74	.79	SAND to SILTY SAND	14	14	50		38.0
6.600	21.65	47.29	.77	SILTY SAND to SANDY SILT	16	15	45		37.0
6.750	22.15	39.03	.75	SILTY SAND to SANDY SILT	13	13	39		36.0
6.900	22.64	34.99	.76	SILTY SAND to SANDY SILT	12	11	36		35.5
7.050	23.13	39.01	.77	SILTY SAND to SANDY SILT	13	13	39		36.0
7.200	23.62	40.13	.81	SILTY SAND to SANDY SILT	13	13	40		36.0
7.350	24.11	41.89	.79	SILTY SAND to SANDY SILT	14	13	41		36.0
7.500	24.61	52.67	.78	SILTY SAND to SANDY SILT	18	17	47		37.0
7.650	25.10	51.26	.82	SILTY SAND to SANDY SILT	17	16	46		37.0
7.800	25.59	48.46	.87	SILTY SAND to SANDY SILT	16	15	44		36.5
7.950	26.08	45.32	.86	SILTY SAND to SANDY SILT	15	14	42		36.5
8.100	26.57	43.25	.87	SILTY SAND to SANDY SILT	14	13	41		36.0
8.250	27.07	48.35	.87	SILTY SAND to SANDY SILT	16	15	44		36.5
8.400	27.56	48.76	.86	SILTY SAND to SANDY SILT	16	15	44		36.5
8.550	28.05	55.09	.80	SILTY SAND to SANDY SILT	18	17	47		37.0
8.700	28.54	53.52	.86	SILTY SAND to SANDY SILT	18	16	46		36.5
8.850	29.04	53.54	.89	SILTY SAND to SANDY SILT	18	16	46		36.5
9.000	29.53	50.67	.90	SILTY SAND to SANDY SILT	17	15	44		36.5
9.150	30.02	54.32	.83	SILTY SAND to SANDY SILT	18	16	46		36.5
9.300	30.51	54.11	.85	SILTY SAND to SANDY SILT	18	16	46		36.5
9.450	31.00	55.30	.87	SILTY SAND to SANDY SILT	18	16	46		36.5
9.600	31.50	58.21	.85	SILTY SAND to SANDY SILT	19	17	48		37.0
9.750	31.99	57.89	.90	SILTY SAND to SANDY SILT	19	17	47		37.0
9.900	32.48	61.44	.91	SILTY SAND to SANDY SILT	20	18	49		37.0
10.050	32.97	73.17	.89	SAND to SILTY SAND	18	16	54		38.0
10.200	33.46	79.14	.88	SAND to SILTY SAND	20	17	56		38.0
10.350	33.96	87.76	.84	SAND to SILTY SAND	22	19	59		38.0
10.500	34.45	97.68	.85	SAND to SILTY SAND	24	21	62		38.5
10.650	34.94	105.29	.83	SAND to SILTY SAND	26	22	64		38.5
10.800	35.43	123.03	.89	SAND to SILTY SAND	31	26	68		39.5
10.950	35.93	114.91	.90	SAND to SILTY SAND	29	24	66		39.0
11.100	36.42	91.20	.90	SAND to SILTY SAND	23	19	59		38.0
11.250	36.91	53.58	2.12	SANDY SILT to CLAYEY SILT	21	18		3.4	
11.400	37.40	62.42	1.28	SILTY SAND to SANDY SILT	21	17	48		36.5
11.550	37.89	22.63	3.05	CLAYEY SILT to SILTY CLAY	11	9		1.4	
11.700	38.39	41.00	1.98	SANDY SILT to CLAYEY SILT	16	13		2.6	
11.850	38.88	50.16	1.44	SILTY SAND to SANDY SILT	17	14	41		35.5
12.000	39.37	63.86	.89	SAND to SILTY SAND	16	13	48		36.5
12.150	39.86	33.57	1.25	SILTY SAND to SANDY SILT	11	9	30		32.0
12.300	40.35	28.91	.77	SILTY SAND to SANDY SILT	10	8	25		31.0
12.450	40.85	11.02	1.38	CLAYEY SILT to SILTY CLAY	6	4		.7	
12.600	41.34	8.21	1.07	SENSITIVE FINE GRAINED	4	3		.6	
12.750	41.83	8.39	1.12	CLAYEY SILT to SILTY CLAY	4	3		.6	
12.900	42.32	8.38	1.18	CLAYEY SILT to SILTY CLAY	4	3		.6	
13.050	42.81	8.50	.89	SENSITIVE FINE GRAINED	4	3		.6	
13.200	43.31	7.91	.78	SENSITIVE FINE GRAINED	4	3		.5	
13.350	43.80	9.00	.51	SENSITIVE FINE GRAINED	5	4		.6	
13.500	44.29	9.48	.57	SENSITIVE FINE GRAINED	5	4		.7	
13.650	44.78	15.07	1.04	SANDY SILT to CLAYEY SILT	6	5		1.0	

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : CPT-05

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
13.800	45.28	116.58	.58	SAND	23	18	64		38.5
13.950	45.77	160.72	.80	SAND	32	25	73		39.5
14.100	46.26	151.24	.96	SAND to SILTY SAND	38	29	71		39.5
14.250	46.75	117.93	1.08	SAND to SILTY SAND	29	23	64		38.5
14.400	47.24	116.74	.40	SAND	23	18	64		38.5
14.550	47.74	88.12	.60	SAND to SILTY SAND	22	17	56		37.5
14.700	48.23	89.06	.44	SAND to SILTY SAND	22	17	56		37.5
14.850	48.72	92.03	.34	SAND to SILTY SAND	23	17	57		37.5
15.000	49.21	88.78	.68	SAND to SILTY SAND	22	17	55		37.5
15.150	49.70	141.98	.53	SAND	28	21	69		39.0
15.300	50.20	143.04	.75	SAND	29	21	69		39.0
15.450	50.69	154.45	.67	SAND	31	23	71		39.0
15.600	51.18	146.89	.68	SAND	29	22	69		39.0
15.750	51.67	154.05	.64	SAND	31	23	71		39.0
15.900	52.17	151.54	.75	SAND	30	22	70		39.0
16.050	52.66	159.14	.57	SAND	32	23	71		39.0
16.200	53.15	152.73	.57	SAND	31	22	70		39.0
16.350	53.64	132.46	2.74	SILTY SAND to SANDY SILT	44	32	66		38.5
16.500	54.13	165.77	3.59	SANDY SILT to CLAYEY SILT	66	48		9.6	
16.650	54.63	79.09	4.92	*VERY STIFF FINE GRAINED	79	57			
16.800	55.12	86.02	4.33	CLAYEY SILT to SILTY CLAY	43	31		4.9	
16.950	55.61	90.97	3.54	SANDY SILT to CLAYEY SILT	36	26		5.2	
17.100	56.10	136.99	5.11	*VERY STIFF FINE GRAINED	100	98			
17.250	56.59	189.06	4.45	*VERY STIFF FINE GRAINED	100	100			
17.400	57.09	264.22	1.71	SAND to SILTY SAND	66	47	85		42.0
17.550	57.58	262.35	2.00	SAND to SILTY SAND	66	47	85		42.0
17.700	58.07	231.67	3.05	SILTY SAND to SANDY SILT	77	55	81		41.0
17.850	58.56	234.12	1.97	SAND to SILTY SAND	59	41	81		41.0
18.000	59.06	397.85	1.78	SAND to SILTY SAND	99	70	96		43.5
18.150	59.55	417.08	2.01	SAND to SILTY SAND	100	73	98		44.0
18.300	60.04	433.50	1.60	SAND to SILTY SAND	100	76	99		44.0
18.450	60.53	384.02	2.18	SAND to SILTY SAND	96	67	95		43.5
18.600	61.02	460.52	1.76	SAND to SILTY SAND	100	80	100		44.0
18.750	61.52	564.11	2.22	*SAND to CLAYEY SAND	100	100			
18.900	62.01	210.26	4.27	*VERY STIFF FINE GRAINED	100	100			
19.050	62.50	129.08	4.60	*VERY STIFF FINE GRAINED	100	89			
19.200	62.99	178.05	5.45	*VERY STIFF FINE GRAINED	100	100			
19.350	63.48	239.45	2.79	SILTY SAND to SANDY SILT	80	55	81		41.0
19.500	63.98	271.13	1.57	SAND to SILTY SAND	68	46	85		42.0
19.650	64.47	249.82	1.79	SAND to SILTY SAND	62	42	82		41.0
19.800	64.96	211.56	3.28	SANDY SILT to CLAYEY SILT	85	57		12.2	
19.950	65.45	47.48	3.99	CLAYEY SILT to SILTY CLAY	24	16		2.6	
20.100	65.94	37.14	4.26	CLAY to SILTY CLAY	25	17		2.0	
20.250	66.44	58.49	5.48	CLAY	58	39		3.2	
20.400	66.93	80.77	5.15	*VERY STIFF FINE GRAINED	81	54			
20.550	67.42	88.80	5.13	*VERY STIFF FINE GRAINED	89	59			
20.700	67.91	298.64	1.03	SAND	60	40	87		42.0
20.850	68.41	295.43	1.40	SAND	59	39	86		42.0
21.000	68.90	258.97	1.70	SAND to SILTY SAND	65	43	82		41.0
21.150	69.39	286.40	1.42	SAND to SILTY SAND	72	47	85		41.5

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : CPT-05

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
21.300	69.88	330.76	1.45	SAND	66	44	89		42.0
21.450	70.37	342.11	2.19	SAND to SILTY SAND	86	56	90		42.5
21.600	70.87	417.25	2.18	SAND to SILTY SAND	100	68	96		43.0
21.750	71.36	197.56	3.89	*SAND to CLAYEY SAND	99	65			

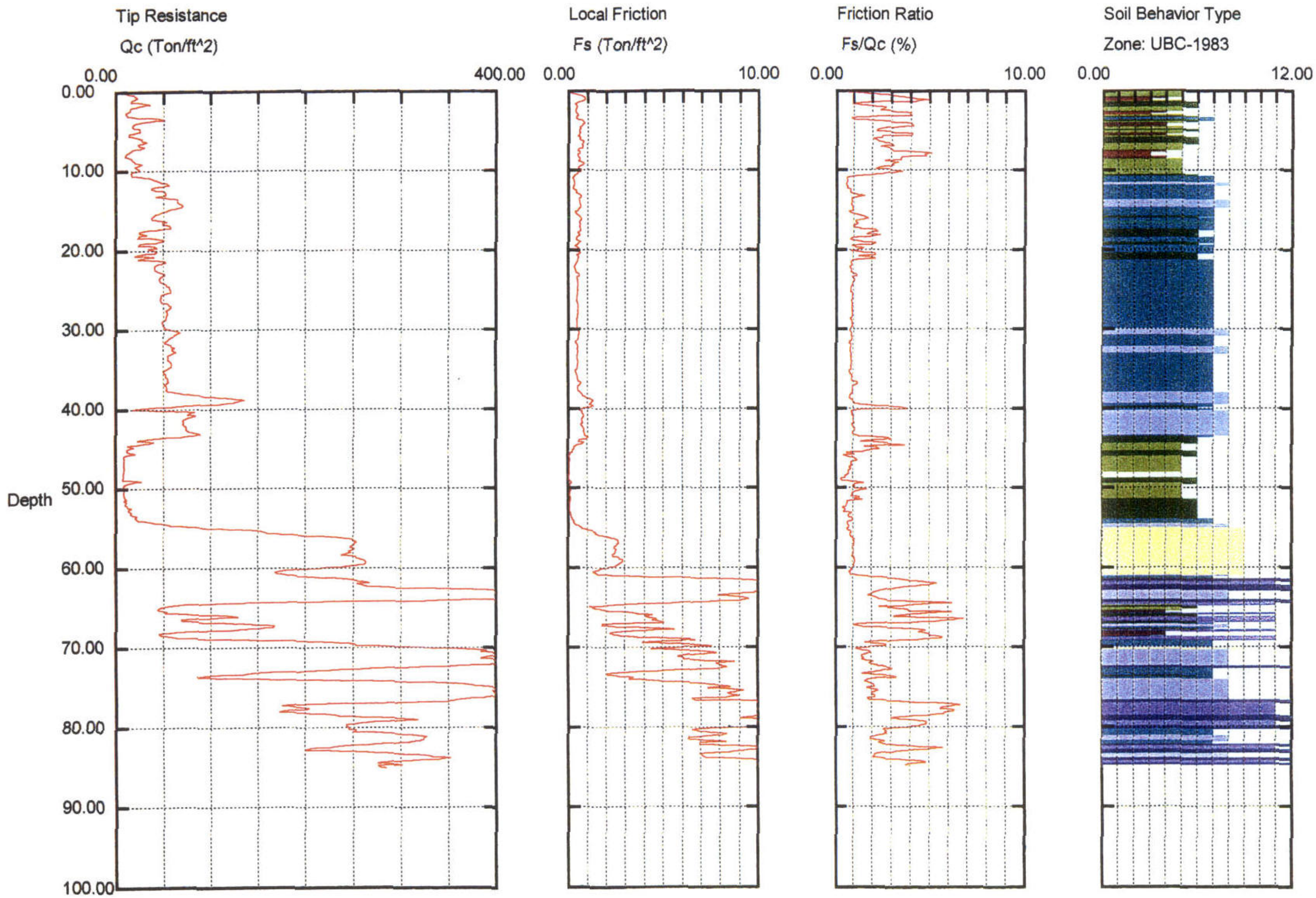
TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

Hushmand Associates

Operator: ALAMEDA NAS #2	CPT Date/Time: 02-21-02 08:15
Sounding: SDF128	Location: CPT-06Seis
Cone Used: 472/GO-VO/R#4	Job Number: 010810



Maximum Depth = 85.14 feet

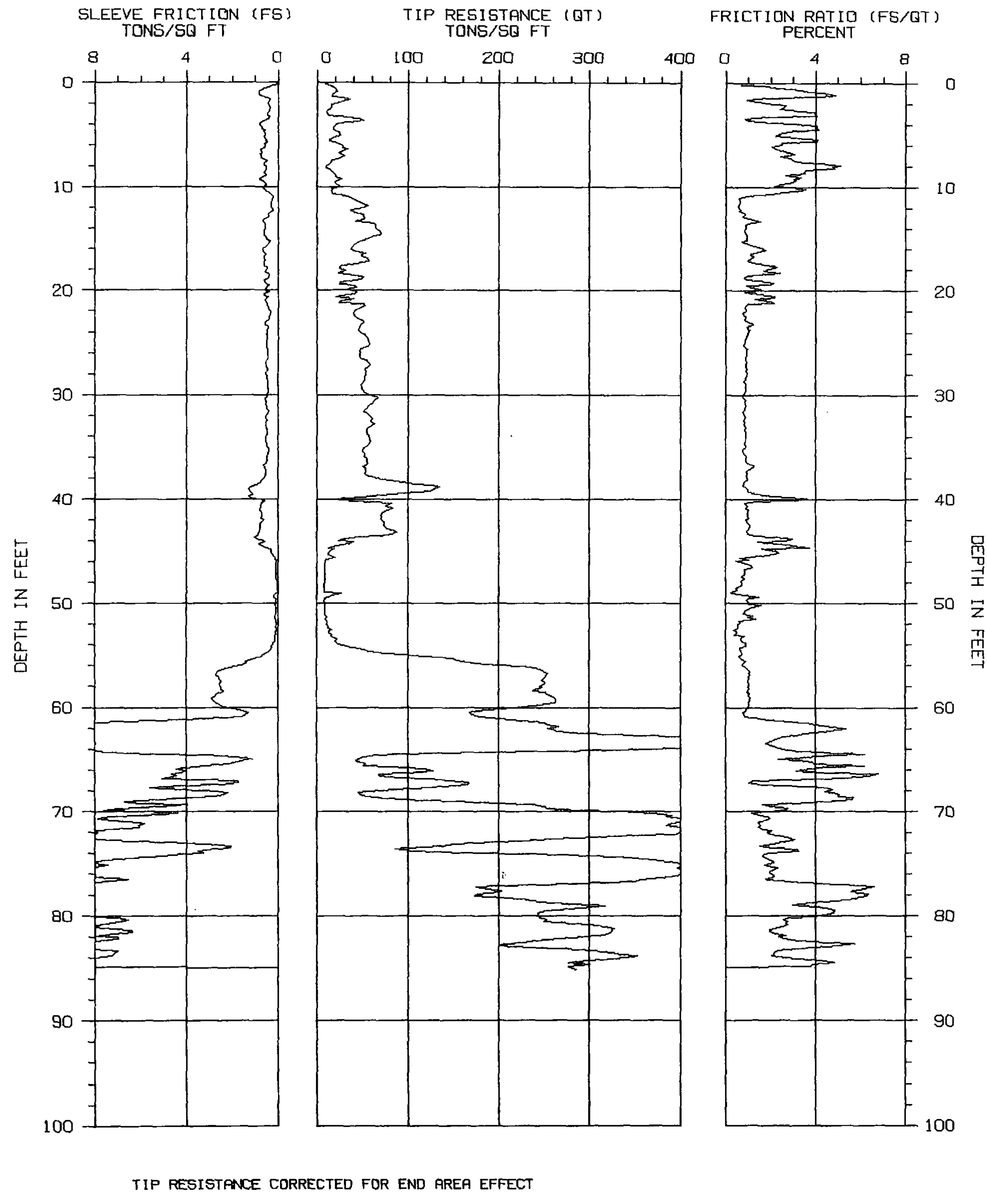
Depth Increment = 0.16 feet

1 sensitive fine grained
organic material
clay

4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt

7 silty sand to sandy silt
8 sand to silty sand
9 sand

10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)



CONE PENETRATION TEST

SOUNDING NUMBER: CPT-06SEIS

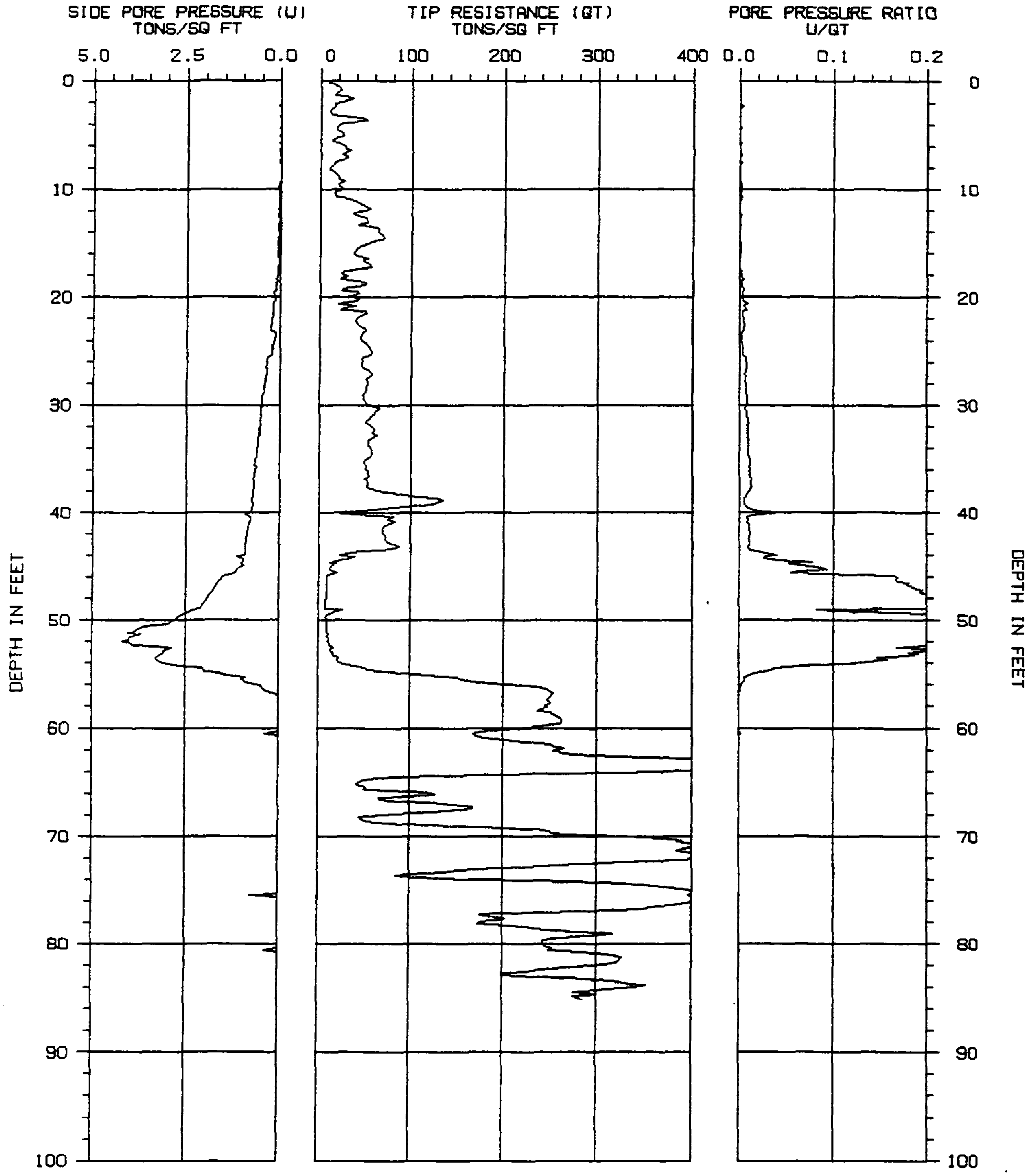
PROJECT NAME : ALAMEDA NAS #2

CONE/RIG : 472/GO-VQ/R#4

PROJECT NUMBER : 010810

DATE/TIME: 02-21-02 08:15

HFA



TIP RESISTANCE CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST		SOUNDING NUMBER: CPT-06SEIS
PROJECT NAME : ALAMEDA NAS #2	CONE/RIG : 472/GO-VO/R#4	HFA
PROJECT NUMBER : 010810	DATE/TIME: 02-21-02 08:15	

CPT INTERPRETATIONS

SOUNDING : CPT-06Seis
PROJECT : ALAMEDA NAS #2
DATE/TIME: 02-21-02 08:15

PROJECT No.: 010810
CONE/RIG : 472/GO-VO/R#4

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	19.40	2.37	CLAYEY SILT to SILTY CLAY	10	16		1.3	
.300	.98	20.12	4.27	CLAY	20	32		1.3	
.450	1.48	27.60	1.81	SANDY SILT to CLAYEY SILT	11	18		2.2	
.600	1.97	21.20	2.17	CLAYEY SILT to SILTY CLAY	11	17		1.7	
.750	2.46	15.23	2.43	CLAYEY SILT to SILTY CLAY	8	12		1.2	
.900	2.95	11.92	3.94	CLAY	12	19		.8	
1.050	3.44	45.80	.85	SILTY SAND to SANDY SILT	15	24	54		44.5
1.200	3.94	25.18	3.14	CLAYEY SILT to SILTY CLAY	13	20		1.7	
1.350	4.43	17.63	4.14	CLAY	18	28		1.2	
1.500	4.92	24.77	2.26	SANDY SILT to CLAYEY SILT	10	16		1.6	
1.650	5.41	13.96	4.08	CLAY	14	22		.9	
1.800	5.91	24.45	2.49	CLAYEY SILT to SILTY CLAY	12	20		1.6	
1.950	6.40	33.23	2.32	SANDY SILT to CLAYEY SILT	13	21		2.2	
2.100	6.89	25.66	3.08	CLAYEY SILT to SILTY CLAY	13	20		1.7	
2.250	7.38	18.36	3.00	CLAYEY SILT to SILTY CLAY	9	14		1.2	
2.400	7.87	12.13	5.11	CLAY	12	18		.8	
2.550	8.37	14.85	3.57	CLAY to SILTY CLAY	10	14		1.0	
2.700	8.86	20.23	2.67	CLAYEY SILT to SILTY CLAY	10	14		1.3	
2.850	9.35	24.77	3.11	CLAYEY SILT to SILTY CLAY	12	17		1.6	
3.000	9.84	24.90	2.21	SANDY SILT to CLAYEY SILT	10	13		1.6	
3.150	10.33	16.25	3.14	CLAY to SILTY CLAY	11	14		1.0	
3.300	10.83	30.06	.93	SILTY SAND to SANDY SILT	10	13	39		37.0
3.450	11.32	44.19	.63	SILTY SAND to SANDY SILT	15	18	50		38.5
3.600	11.81	56.02	.55	SAND to SILTY SAND	14	17	56		39.0
3.750	12.30	37.48	.61	SILTY SAND to SANDY SILT	12	15	44		37.5
3.900	12.80	51.90	.79	SILTY SAND to SANDY SILT	17	20	53		38.5
4.050	13.29	42.49	1.58	SILTY SAND to SANDY SILT	14	16	46		38.0
4.200	13.78	64.39	.89	SAND to SILTY SAND	16	18	58		39.0
4.350	14.27	69.05	.87	SAND to SILTY SAND	17	19	59		39.0
4.500	14.76	62.42	.95	SILTY SAND to SANDY SILT	21	23	56		38.5
4.650	15.26	48.14	.73	SILTY SAND to SANDY SILT	16	17	48		38.0
4.800	15.75	38.88	1.41	SILTY SAND to SANDY SILT	13	14	42		37.0
4.950	16.24	44.02	1.52	SILTY SAND to SANDY SILT	15	16	45		37.5
5.100	16.73	49.22	1.24	SILTY SAND to SANDY SILT	16	17	48		38.0
5.250	17.22	56.89	1.05	SILTY SAND to SANDY SILT	19	20	52		38.0
5.400	17.72	24.30	2.26	SANDY SILT to CLAYEY SILT	10	10		1.6	
5.550	18.21	24.22	2.39	CLAYEY SILT to SILTY CLAY	12	12		1.5	
5.700	18.70	51.03	.80	SILTY SAND to SANDY SILT	17	17	48		37.5
5.850	19.19	25.96	2.16	SANDY SILT to CLAYEY SILT	10	11		1.7	
6.000	19.69	43.38	1.15	SILTY SAND to SANDY SILT	14	15	43		37.0
6.150	20.18	43.53	.87	SILTY SAND to SANDY SILT	15	15	43		36.5

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : CPT-06Seis

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
6.300	20.67	20.56	2.19	CLAYEY SILT to SILTY CLAY	10	10		1.5	
6.450	21.16	24.01	2.17	SANDY SILT to CLAYEY SILT	10	9		1.5	
6.600	21.65	48.46	.87	SILTY SAND to SANDY SILT	16	16	46		37.0
6.750	22.15	40.64	.79	SILTY SAND to SANDY SILT	14	13	40		36.0
6.900	22.64	43.79	.89	SILTY SAND to SANDY SILT	15	14	42		36.5
7.050	23.13	50.92	1.16	SILTY SAND to SANDY SILT	17	16	47		37.0
7.200	23.62	45.66	1.01	SILTY SAND to SANDY SILT	15	15	43		36.5
7.350	24.11	49.69	1.01	SILTY SAND to SANDY SILT	17	16	46		37.0
7.500	24.61	56.21	.89	SILTY SAND to SANDY SILT	19	18	49		37.5
7.650	25.10	57.51	.87	SILTY SAND to SANDY SILT	19	18	49		37.5
7.800	25.59	48.12	.98	SILTY SAND to SANDY SILT	16	15	44		36.5
7.950	26.08	46.93	.94	SILTY SAND to SANDY SILT	16	15	43		36.5
8.100	26.57	48.71	.90	SILTY SAND to SANDY SILT	16	15	44		36.5
8.250	27.07	58.15	.89	SILTY SAND to SANDY SILT	19	18	49		37.5
8.400	27.56	52.92	.94	SILTY SAND to SANDY SILT	18	16	46		37.0
8.550	28.05	53.71	.95	SILTY SAND to SANDY SILT	18	16	46		37.0
8.700	28.54	52.56	.89	SILTY SAND to SANDY SILT	18	16	46		36.5
8.850	29.04	48.27	.91	SILTY SAND to SANDY SILT	16	14	43		36.5
9.000	29.53	49.90	.92	SILTY SAND to SANDY SILT	17	15	44		36.5
9.150	30.02	57.62	.82	SAND to SILTY SAND	14	13	48		37.0
9.300	30.51	63.25	.89	SAND to SILTY SAND	16	14	50		37.5
9.450	31.00	57.89	.90	SILTY SAND to SANDY SILT	19	17	48		37.0
9.600	31.50	52.84	.87	SILTY SAND to SANDY SILT	18	15	45		36.5
9.750	31.99	58.36	.87	SILTY SAND to SANDY SILT	19	17	48		37.0
9.900	32.48	59.36	.83	SAND to SILTY SAND	15	13	48		37.0
10.050	32.97	60.08	.87	SAND to SILTY SAND	15	13	48		37.0
10.200	33.46	55.13	.87	SILTY SAND to SANDY SILT	18	16	46		36.5
10.350	33.96	56.68	.88	SILTY SAND to SANDY SILT	19	16	46		36.5
10.500	34.45	58.87	.88	SILTY SAND to SANDY SILT	20	17	47		36.5
10.650	34.94	54.60	.88	SILTY SAND to SANDY SILT	18	15	45		36.5
10.800	35.43	50.22	.78	SILTY SAND to SANDY SILT	17	14	42		36.0
10.950	35.93	51.86	.93	SILTY SAND to SANDY SILT	17	15	43		36.0
11.100	36.42	54.47	.92	SILTY SAND to SANDY SILT	18	15	44		36.0
11.250	36.91	50.33	1.27	SILTY SAND to SANDY SILT	17	14	42		36.0
11.400	37.40	53.92	.96	SILTY SAND to SANDY SILT	18	15	44		36.0
11.550	37.89	62.12	.93	SILTY SAND to SANDY SILT	21	17	48		36.5
11.700	38.39	94.09	.81	SAND to SILTY SAND	24	19	60		38.0
11.850	38.88	134.93	.88	SAND to SILTY SAND	34	28	70		39.5
12.000	39.37	109.81	1.07	SAND to SILTY SAND	27	22	64		38.5
12.150	39.86	34.42	3.43	CLAYEY SILT to SILTY CLAY	17	14		2.1	
12.300	40.35	82.58	.87	SAND to SILTY SAND	21	17	55		37.5
12.450	40.85	83.70	.92	SAND to SILTY SAND	21	17	56		38.0
12.600	41.34	70.94	.99	SAND to SILTY SAND	18	14	51		37.0
12.750	41.83	70.32	1.05	SAND to SILTY SAND	18	14	50		37.0
12.900	42.32	74.04	1.00	SAND to SILTY SAND	19	15	52		37.0
13.050	42.81	76.48	1.03	SAND to SILTY SAND	19	15	53		37.0
13.200	43.31	83.17	1.01	SAND to SILTY SAND	21	16	55		37.5
13.350	43.80	31.65	3.00	CLAYEY SILT to SILTY CLAY	16	12		1.9	
13.500	44.29	35.73	2.35	SANDY SILT to CLAYEY SILT	14	11		2.2	
13.650	44.78	19.02	1.63	SANDY SILT to CLAYEY SILT	8	6		1.3	

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

SOUNDING : CPT-06Seis

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
13.800	45.28	12.30	1.87	CLAYEY SILT to SILTY CLAY	6	5		.8	
13.950	45.77	11.52	1.04	CLAYEY SILT to SILTY CLAY	6	4		.9	
14.100	46.26	9.54	1.05	CLAYEY SILT to SILTY CLAY	5	4		.7	
14.250	46.75	9.37	.85	CLAYEY SILT to SILTY CLAY	5	4		.7	
14.400	47.24	9.13	.88	SENSITIVE FINE GRAINED	5	3		.6	
14.550	47.74	9.19	.76	SENSITIVE FINE GRAINED	5	4		.6	
14.700	48.23	8.36	.72	SENSITIVE FINE GRAINED	4	3		.6	
14.850	48.72	8.12	.49	SENSITIVE FINE GRAINED	4	3		.5	
15.000	49.21	18.71	.86	SANDY SILT to CLAYEY SILT	7	6		1.3	
15.150	49.70	8.53	.94	SENSITIVE FINE GRAINED	4	3		.6	
15.300	50.20	9.26	1.62	CLAYEY SILT to SILTY CLAY	5	3		.5	
15.450	50.69	9.27	.86	CLAYEY SILT to SILTY CLAY	5	3		.6	
15.600	51.18	10.53	1.23	CLAYEY SILT to SILTY CLAY	5	4		.6	
15.750	51.67	11.97	.58	SANDY SILT to CLAYEY SILT	5	4		.9	
15.900	52.17	12.88	.54	SANDY SILT to CLAYEY SILT	5	4		1.0	
16.050	52.66	13.78	.51	SANDY SILT to CLAYEY SILT	6	4		1.1	
16.200	53.15	16.05	.87	SANDY SILT to CLAYEY SILT	6	5		1.3	
16.350	53.64	20.51	.73	SANDY SILT to CLAYEY SILT	8	6		1.4	
16.500	54.13	32.41	.89	SILTY SAND to SANDY SILT	11	8	26		30.5
16.650	54.63	57.34	.68	SAND to SILTY SAND	14	10	42		34.5
16.800	55.12	118.23	.60	SAND	24	17	62		38.0
16.950	55.61	159.08	.89	SAND	32	23	71		39.0
17.100	56.10	226.72	.85	SAND	45	33	81		41.0
17.250	56.59	251.26	1.06	SAND	50	36	84		42.0
17.400	57.09	251.41	1.03	SAND	50	36	84		42.0
17.550	57.58	251.22	.98	SAND	50	36	84		42.0
17.700	58.07	247.46	1.01	SAND	49	35	83		41.5
17.850	58.56	252.43	.94	SAND	50	36	84		41.5
18.000	59.06	261.91	1.11	SAND	52	37	84		42.0
18.150	59.55	261.63	1.04	SAND	52	37	84		42.0
18.300	60.04	215.83	.99	SAND	43	30	79		40.5
18.450	60.53	167.56	.76	SAND	34	23	71		39.0
18.600	61.02	190.48	1.22	SAND to SILTY SAND	48	33	75		39.5
18.750	61.52	250.50	3.64	*SAND to CLAYEY SAND	100	87			
18.900	62.01	254.17	5.38	*VERY STIFF FINE GRAINED	100	100			
19.050	62.50	293.84	3.45	*SAND to CLAYEY SAND	100	100			
19.200	62.99	471.72	2.35	*SAND to CLAYEY SAND	100	100			
19.350	63.48	440.70	1.80	SAND to SILTY SAND	100	75	99		44.0
19.500	63.98	402.70	2.36	SAND to SILTY SAND	100	69	96		43.5
19.650	64.47	88.78	6.20	*VERY STIFF FINE GRAINED	89	60			
19.800	64.96	46.67	2.38	SANDY SILT to CLAYEY SILT	19	13		2.9	
19.950	65.45	53.47	3.95	CLAYEY SILT to SILTY CLAY	27	18		2.9	
20.100	65.94	117.44	3.76	SANDY SILT to CLAYEY SILT	47	32		6.7	
20.250	66.44	68.26	6.81	*VERY STIFF FINE GRAINED	68	46			
20.400	66.93	123.18	4.09	*VERY STIFF FINE GRAINED	100	83			
20.550	67.42	165.07	1.50	SAND to SILTY SAND	41	28	70		38.5
20.700	67.91	85.98	4.78	*VERY STIFF FINE GRAINED	86	57			
20.850	68.41	47.55	5.05	CLAY	48	32		2.6	
21.000	68.90	93.41	5.63	*VERY STIFF FINE GRAINED	93	62			
21.150	69.39	236.18	1.67	SAND to SILTY SAND	59	39	80		40.0

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

SOUNDING : CPT-06Seis

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
21.300	69.88	287.12	2.65	SILTY SAND to SANDY SILT	96	63	85		41.5
21.450	70.37	390.50	1.51	SAND	78	51	94		43.0
21.600	70.87	412.77	1.75	SAND to SILTY SAND	100	68	95		43.0
21.750	71.36	384.68	1.58	SAND to SILTY SAND	96	63	93		43.0
21.900	71.85	421.03	2.08	SAND to SILTY SAND	100	69	96		43.0
22.050	72.34	352.01	2.31	SAND to SILTY SAND	88	57	91		42.5
22.200	72.83	227.23	2.83	SILTY SAND to SANDY SILT	76	49	78		39.5
22.350	73.33	136.20	1.51	SAND to SILTY SAND	34	22	63		38.0
22.500	73.82	105.27	3.27	SANDY SILT to CLAYEY SILT	42	27		5.9	
22.650	74.31	305.54	1.64	SAND to SILTY SAND	76	49	86		42.0
22.800	74.80	377.56	2.10	SAND to SILTY SAND	94	60	92		42.5
22.950	75.30	418.33	2.08	SAND to SILTY SAND	100	67	95		43.0
23.100	75.79	424.41	1.98	SAND to SILTY SAND	100	68	95		43.0
23.250	76.28	388.25	2.15	SAND to SILTY SAND	97	62	93		42.5
23.400	76.77	350.01	2.85	*SAND to CLAYEY SAND	100	100			
23.550	77.26	175.25	6.62	*VERY STIFF FINE GRAINED	100	100			
23.700	77.76	200.70	5.62	*VERY STIFF FINE GRAINED	100	100			
23.850	78.25	204.33	5.78	*VERY STIFF FINE GRAINED	100	100			
24.000	78.74	257.91	3.79	*SAND to CLAYEY SAND	100	81			
24.150	79.23	290.14	4.43	*VERY STIFF FINE GRAINED	100	100			
24.300	79.72	242.68	4.78	*VERY STIFF FINE GRAINED	100	100			
24.450	80.22	245.57	2.84	SILTY SAND to SANDY SILT	82	51	79		39.5
24.600	80.71	281.58	2.66	SILTY SAND to SANDY SILT	94	58	83		40.5
24.750	81.20	327.64	2.16	SAND to SILTY SAND	82	51	87		41.5
24.900	81.69	320.50	2.22	SAND to SILTY SAND	80	50	87		41.5
25.050	82.19	267.62	2.60	SILTY SAND to SANDY SILT	89	55	81		40.0
25.200	82.68	206.03	5.75	*VERY STIFF FINE GRAINED	100	100			
25.350	83.17	272.00	3.14	*SAND to CLAYEY SAND	100	83			
25.500	83.66	334.50	2.14	SAND to SILTY SAND	84	51	88		41.5
25.650	84.15	312.72	3.80	*SAND to CLAYEY SAND	100	95			
25.800	84.65	301.06	3.89	*SAND to CLAYEY SAND	100	92			
25.950	85.14	285.15	*****		0	0			.0

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

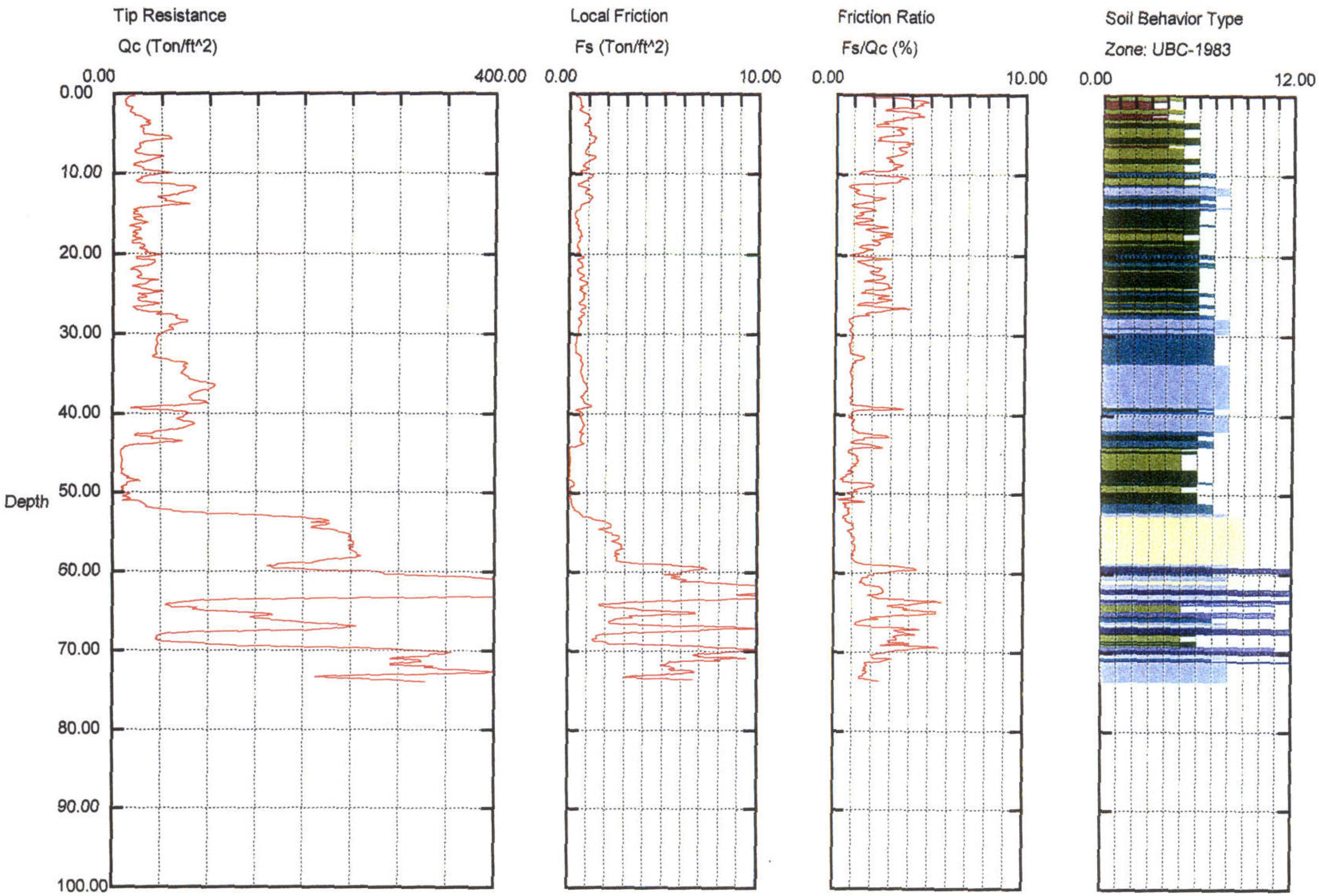
HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

Hushmand Associates

Operator: ALAMEDA NAS #2
Sounding: SDF125
Cone Used: 408/GO-VO/R#4

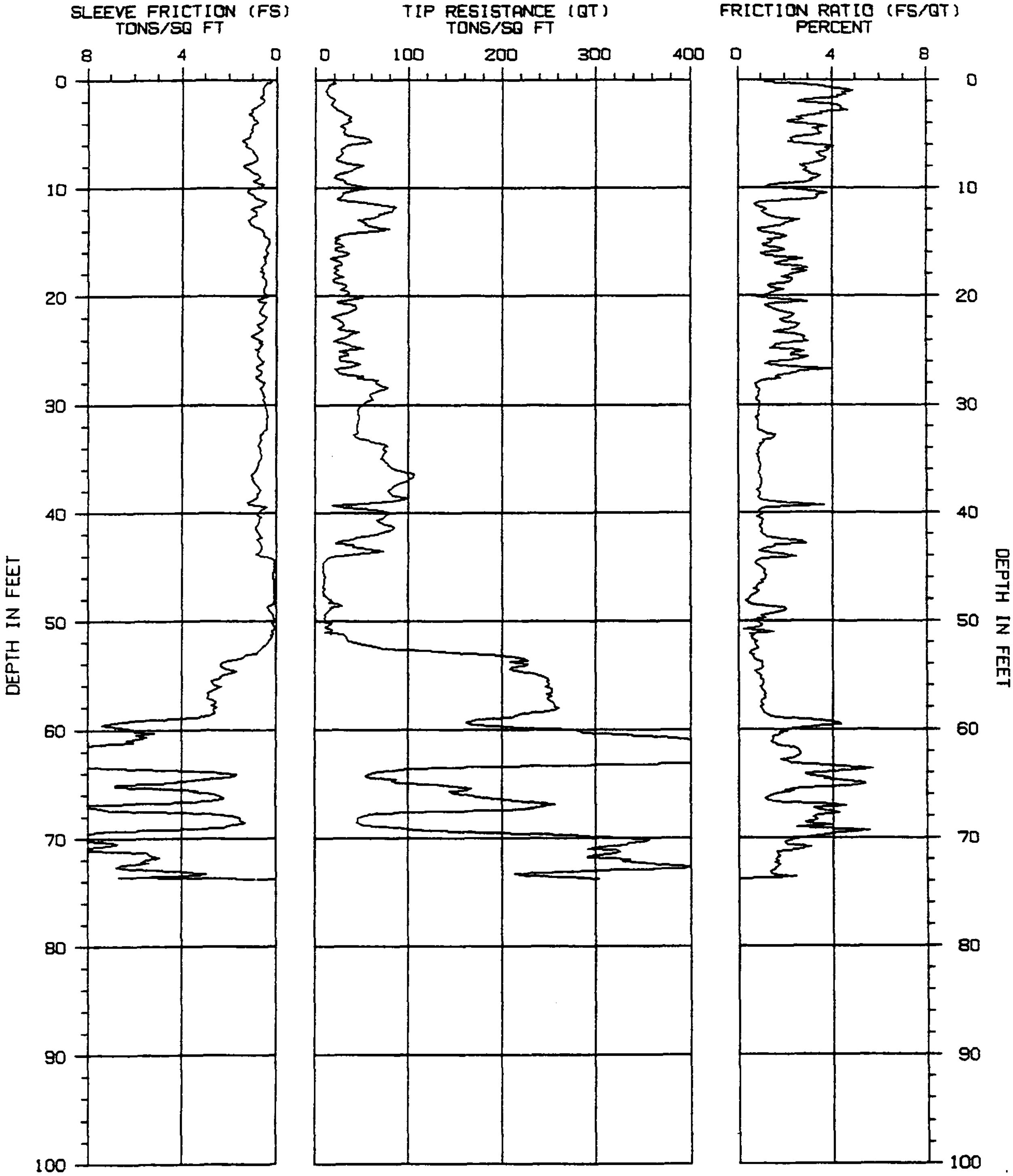
CPT Date/Time: 02-20-02 11:41
Location: CPT-07
Job Number: 010810



Maximum Depth = 73.98 feet

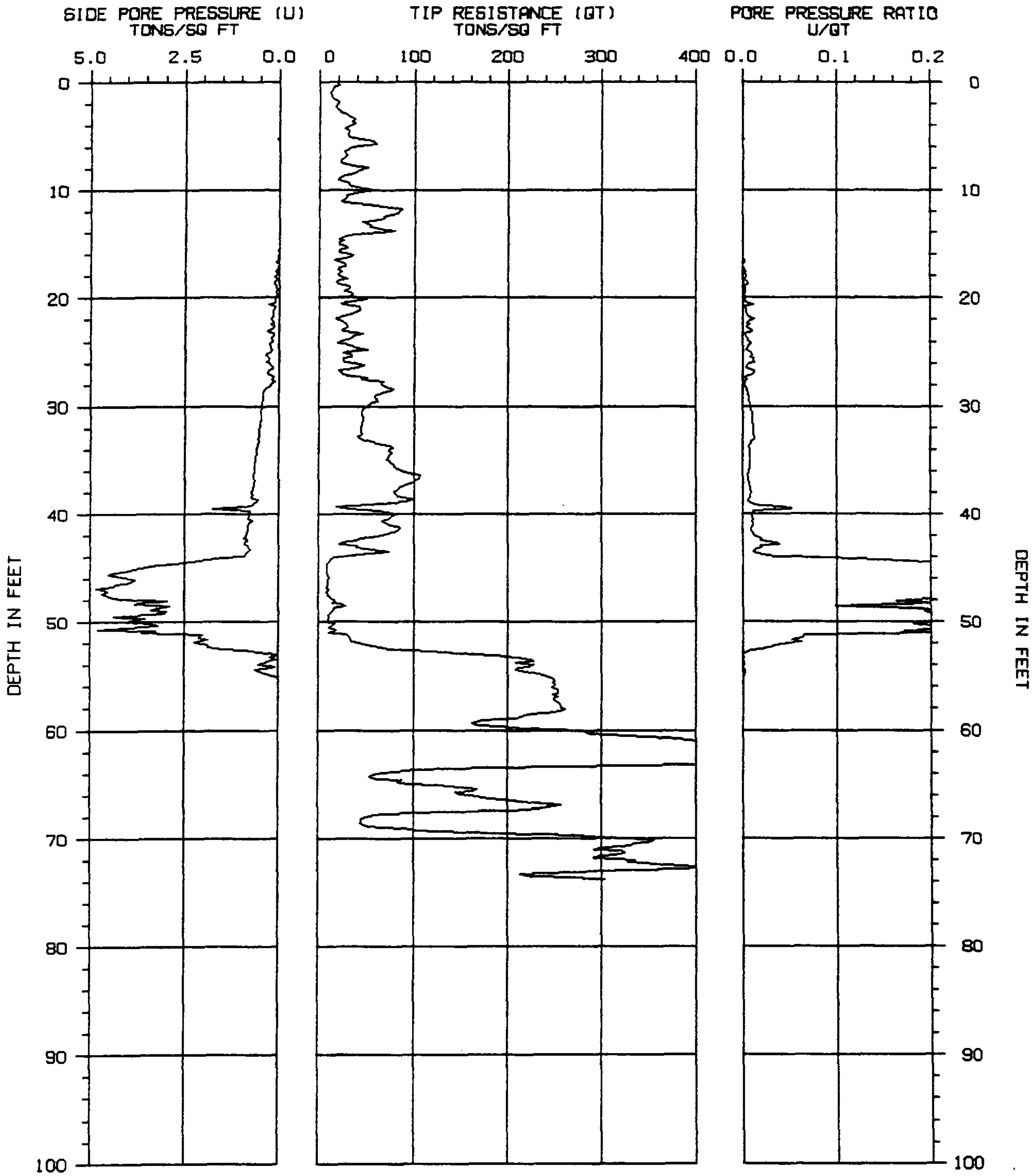
Depth Increment = 0.16 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)



TIP RESISTANCE CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST		SOUNDING NUMBER: CPT-07
PROJECT NAME : ALAMEDA NAS #2	CONE/RIG : 408/GO-VO/R#4	HFA
PROJECT NUMBER : 010810	DATE/TIME: 02-20-02 11:41	



TIP RESISTANCE CORRECTED FOR END AREA EFFECT

CONE PENETRATION TEST

SOUNDING NUMBER: CPT-07

PROJECT NAME : ALAMEDA NAS #2	CONE/RIG : 408/GO-VO/R#4
PROJECT NUMBER : 010810	DATE/TIME: 02-20-02 11:41

HFA

CPT INTERPRETATIONS

SOUNDING : CPT-07
PROJECT : ALAMEDA NAS #2
DATE/TIME: 02-20-02 11:41

PROJECT No.: 010810
CONE/RIG : 408/GO-VO/R#4

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
.150	.49	15.19	3.29	CLAY to SILTY CLAY	10	16		1.0	
.300	.98	11.68	4.88	CLAY	12	19		.8	
.450	1.48	13.21	4.24	CLAY	13	21		.9	
.600	1.97	19.65	2.60	CLAYEY SILT to SILTY CLAY	10	16		1.3	
.750	2.46	19.29	4.41	CLAY	19	31		1.3	
.900	2.95	29.23	3.52	CLAYEY SILT to SILTY CLAY	15	23		1.9	
1.050	3.44	38.03	2.52	SANDY SILT to CLAYEY SILT	15	24		2.5	
1.200	3.94	36.41	2.17	SANDY SILT to CLAYEY SILT	15	23		2.4	
1.350	4.43	31.68	3.22	CLAYEY SILT to SILTY CLAY	16	25		2.1	
1.500	4.92	31.40	3.57	CLAYEY SILT to SILTY CLAY	16	25		2.1	
1.650	5.41	56.79	2.25	SANDY SILT to CLAYEY SILT	23	36		3.8	
1.800	5.91	41.94	3.00	CLAYEY SILT to SILTY CLAY	21	34		2.8	
1.950	6.40	27.11	3.87	CLAY to SILTY CLAY	18	29		1.8	
2.100	6.89	27.85	3.41	CLAYEY SILT to SILTY CLAY	14	22		1.8	
2.250	7.38	23.01	3.65	CLAY to SILTY CLAY	15	24		1.5	
2.400	7.87	51.86	2.66	SANDY SILT to CLAYEY SILT	21	31		3.4	
2.550	8.37	38.71	2.82	SANDY SILT to CLAYEY SILT	15	22		2.5	
2.700	8.86	21.92	3.51	CLAY to SILTY CLAY	15	20		1.4	
2.850	9.35	31.12	3.18	CLAYEY SILT to SILTY CLAY	16	21		2.0	
3.000	9.84	53.03	1.23	SILTY SAND to SANDY SILT	18	23	57		39.5
3.150	10.33	34.59	3.50	CLAYEY SILT to SILTY CLAY	17	22		2.3	
3.300	10.83	27.32	3.40	CLAYEY SILT to SILTY CLAY	14	17		1.8	
3.450	11.32	51.43	.89	SILTY SAND to SANDY SILT	17	21	54		39.0
3.600	11.81	86.34	1.10	SAND to SILTY SAND	22	26	68		42.0
3.750	12.30	75.70	1.10	SAND to SILTY SAND	19	23	64		40.5
3.900	12.80	58.36	1.92	SILTY SAND to SANDY SILT	19	23	56		39.0
4.050	13.29	54.98	1.98	SILTY SAND to SANDY SILT	18	21	54		38.5
4.200	13.78	79.26	.85	SAND to SILTY SAND	20	22	64		40.0
4.350	14.27	31.91	1.72	SANDY SILT to CLAYEY SILT	13	14		2.5	
4.500	14.76	25.41	1.30	SANDY SILT to CLAYEY SILT	10	11		2.0	
4.650.	15.26	28.13	1.07	SILTY SAND to SANDY SILT	9	10	33		35.0
4.800	15.75	21.97	2.00	SANDY SILT to CLAYEY SILT	9	9		1.7	
4.950	16.24	32.33	1.24	SILTY SAND to SANDY SILT	11	11	36		36.0
5.100	16.73	24.01	1.96	SANDY SILT to CLAYEY SILT	10	10		1.8	
5.250	17.22	21.97	2.69	CLAYEY SILT to SILTY CLAY	11	11		1.4	
5.400	17.72	22.82	2.94	CLAYEY SILT to SILTY CLAY	11	12		1.5	
5.550	18.21	29.98	1.90	SANDY SILT to CLAYEY SILT	12	12		2.3	
5.700	18.70	20.69	2.22	CLAYEY SILT to SILTY CLAY	10	11		1.6	
5.850	19.19	31.74	1.48	SANDY SILT to CLAYEY SILT	13	13		2.5	
6.000	19.69	36.01	1.50	SILTY SAND to SANDY SILT	12	12	38		36.0
6.150	20.18	50.58	.97	SILTY SAND to SANDY SILT	17	17	47		37.5

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : CPT-07

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
6.300	20.67	25.01	1.72	SANDY SILT to CLAYEY SILT	10	10		1.9	
6.450	21.16	42.28	1.56	SILTY SAND to SANDY SILT	14	14	42		36.5
6.600	21.65	26.41	2.42	CLAYEY SILT to SILTY CLAY	13	13		1.7	
6.750	22.15	25.09	1.83	SANDY SILT to CLAYEY SILT	10	10		1.9	
6.900	22.64	30.06	2.63	CLAYEY SILT to SILTY CLAY	15	15		1.9	
7.050	23.13	27.26	2.09	SANDY SILT to CLAYEY SILT	11	11		1.7	
7.200	23.62	35.65	2.81	CLAYEY SILT to SILTY CLAY	18	17		2.3	
7.350	24.11	19.91	2.96	CLAYEY SILT to SILTY CLAY	10	9		1.2	
7.500	24.61	40.81	1.62	SILTY SAND to SANDY SILT	14	13	40		36.0
7.650	25.10	26.22	2.82	CLAYEY SILT to SILTY CLAY	13	12		1.7	
7.800	25.59	26.22	2.97	CLAYEY SILT to SILTY CLAY	13	12		1.6	
7.950	26.08	38.45	1.35	SILTY SAND to SANDY SILT	13	12	38		35.5
8.100	26.57	29.81	2.88	CLAYEY SILT to SILTY CLAY	15	14		1.9	
8.250	27.07	25.43	2.44	CLAYEY SILT to SILTY CLAY	13	12		1.6	
8.400	27.56	45.74	1.84	SANDY SILT to CLAYEY SILT	18	17		2.9	
8.550	28.05	65.94	.76	SAND to SILTY SAND	16	15	52		38.0
8.700	28.54	76.95	.88	SAND to SILTY SAND	19	17	57		38.0
8.850	29.04	59.12	.91	SILTY SAND to SANDY SILT	20	18	49		37.0
9.000	29.53	62.84	.83	SAND to SILTY SAND	16	14	51		37.5
9.150	30.02	51.28	.92	SILTY SAND to SANDY SILT	17	15	45		36.5
9.300	30.51	45.53	.90	SILTY SAND to SANDY SILT	15	13	41		36.0
9.450	31.00	46.46	.88	SILTY SAND to SANDY SILT	15	14	41		36.0
9.600	31.50	45.66	.85	SILTY SAND to SANDY SILT	15	13	41		36.0
9.750	31.99	44.95	.87	SILTY SAND to SANDY SILT	15	13	40		36.0
9.900	32.48	45.93	1.11	SILTY SAND to SANDY SILT	15	13	41		36.0
10.050	32.97	43.13	1.58	SILTY SAND to SANDY SILT	14	12	39		35.0
10.200	33.46	60.57	1.12	SILTY SAND to SANDY SILT	20	17	48		37.0
10.350	33.96	74.57	.97	SAND to SILTY SAND	19	16	54		38.0
10.500	34.45	74.80	.91	SAND to SILTY SAND	19	16	54		38.0
10.650	34.94	71.11	.89	SAND to SILTY SAND	18	15	53		37.5
10.800	35.43	79.31	.95	SAND to SILTY SAND	20	17	55		38.0
10.950	35.93	84.96	.95	SAND to SILTY SAND	21	18	57		38.0
11.100	36.42	106.93	.91	SAND to SILTY SAND	27	22	64		38.5
11.250	36.91	101.81	.99	SAND to SILTY SAND	25	21	62		38.5
11.400	37.40	87.25	.97	SAND to SILTY SAND	22	18	58		38.0
11.550	37.89	79.56	.82	SAND to SILTY SAND	20	16	55		38.0
11.700	38.39	83.24	.95	SAND to SILTY SAND	21	17	56		38.0
11.850	38.88	90.50	1.09	SAND to SILTY SAND	23	19	58		38.0
12.000	39.37	19.57	3.63	CLAY to SILTY CLAY	13	11		1.2	
12.150	39.86	76.74	.90	SAND to SILTY SAND	19	16	53		37.5
12.300	40.35	74.31	.81	SAND to SILTY SAND	19	15	52		37.0
12.450	40.85	70.53	1.04	SAND to SILTY SAND	18	14	51		37.0
12.600	41.34	85.85	.97	SAND to SILTY SAND	21	17	56		38.0
12.750	41.83	73.68	1.00	SAND to SILTY SAND	18	15	52		37.0
12.900	42.32	45.29	1.28	SILTY SAND to SANDY SILT	15	12	38		34.0
13.050	42.81	22.73	2.90	CLAYEY SILT to SILTY CLAY	11	9		1.4	
13.200	43.31	56.66	1.06	SILTY SAND to SANDY SILT	19	15	44		36.0
13.350	43.80	43.32	1.92	SANDY SILT to CLAYEY SILT	17	14		2.7	
13.500	44.29	12.93	1.01	SANDY SILT to CLAYEY SILT	5	4		1.0	
13.650	44.78	9.05	.77	SENSITIVE FINE GRAINED	5	4		.6	

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.

SOUNDING : CPT-07

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
13.800	45.28	9.55	1.15	CLAYEY SILT to SILTY CLAY	5	4		.7	
13.950	45.77	10.74	1.21	CLAYEY SILT to SILTY CLAY	5	4		.6	
14.100	46.26	10.22	.98	CLAYEY SILT to SILTY CLAY	5	4		.8	
14.250	46.75	9.64	.83	CLAYEY SILT to SILTY CLAY	5	4		.7	
14.400	47.24	9.84	.91	CLAYEY SILT to SILTY CLAY	5	4		.7	
14.550	47.74	12.55	.56	SANDY SILT to CLAYEY SILT	5	4		1.0	
14.700	48.23	15.80	.51	SANDY SILT to CLAYEY SILT	6	5		1.3	
14.850	48.72	17.10	1.99	CLAYEY SILT to SILTY CLAY	9	6		1.1	
15.000	49.21	12.77	1.64	CLAYEY SILT to SILTY CLAY	6	5		.8	
15.150	49.70	10.22	1.17	CLAYEY SILT to SILTY CLAY	5	4		.6	
15.300	50.20	18.56	.92	SANDY SILT to CLAYEY SILT	7	6		1.3	
15.450	50.69	17.71	.23	SILTY SAND to SANDY SILT	6	4	9		29.0
15.600	51.18	29.91	.50	SILTY SAND to SANDY SILT	10	7	24		30.5
15.750	51.67	34.98	.60	SILTY SAND to SANDY SILT	12	9	28		31.0
15.900	52.17	51.00	.75	SILTY SAND to SANDY SILT	17	13	39		33.5
16.050	52.66	98.75	.69	SAND to SILTY SAND	25	18	58		37.5
16.200	53.15	186.95	.66	SAND	37	27	76		40.0
16.350	53.64	227.66	.86	SAND	46	33	82		41.5
16.500	54.13	224.56	1.02	SAND	45	33	81		41.0
16.650	54.63	219.80	.75	SAND	44	32	80		41.0
16.800	55.12	243.19	.97	SAND	49	35	83		42.0
16.950	55.61	247.63	1.09	SAND	50	36	84		42.0
17.100	56.10	247.01	.94	SAND	49	35	83		42.0
17.250	56.59	253.22	1.13	SAND	51	36	84		42.0
17.400	57.09	248.90	1.16	SAND	50	35	83		42.0
17.550	57.58	255.68	.99	SAND	51	36	84		42.0
17.700	58.07	261.01	.99	SAND	52	37	85		42.0
17.850	58.56	221.99	1.16	SAND	44	31	80		40.5
18.000	59.06	174.14	2.89	SILTY SAND to SANDY SILT	58	41	73		39.0
18.150	59.55	169.00	4.36	*VERY STIFF FINE GRAINED	100	100			
18.300	60.04	279.14	2.11	SAND to SILTY SAND	70	49	86		42.0
18.450	60.53	327.68	1.80	SAND to SILTY SAND	82	57	91		42.5
18.600	61.02	420.73	1.50	SAND	84	58	98		44.0
18.750	61.52	516.84	1.67	SAND to SILTY SAND	100	89	100		44.5
18.900	62.01	567.36	2.57	*SAND to CLAYEY SAND	100	100			
19.050	62.50	498.62	2.54	*SAND to CLAYEY SAND	100	100			
19.200	62.99	494.58	2.14	SAND to SILTY SAND	100	85	100		44.5
19.350	63.48	182.09	4.45	*VERY STIFF FINE GRAINED	100	100			
19.500	63.98	68.07	4.05	CLAYEY SILT to SILTY CLAY	34	23		3.8	
19.650	64.47	61.89	3.88	CLAYEY SILT to SILTY CLAY	31	21		3.4	
19.800	64.96	92.12	5.38	*VERY STIFF FINE GRAINED	92	62			
19.950	65.45	168.30	2.78	SILTY SAND to SANDY SILT	56	38	71		39.0
20.100	65.94	150.07	1.71	SAND to SILTY SAND	38	25	67		38.5
20.250	66.44	196.22	1.16	SAND to SILTY SAND	49	33	75		39.5
20.400	66.93	255.94	3.45	*SAND to CLAYEY SAND	100	86			
20.550	67.42	204.50	3.43	*SAND to CLAYEY SAND	100	68			
20.700	67.91	54.73	3.38	CLAYEY SILT to SILTY CLAY	27	18		3.0	
20.850	68.41	45.93	3.33	CLAYEY SILT to SILTY CLAY	23	15		2.8	
21.000	68.90	51.82	3.98	CLAYEY SILT to SILTY CLAY	26	17		2.8	
21.150	69.39	127.81	5.55	*VERY STIFF FINE GRAINED	100	84			

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

Interpretations based on: Robertson and Campanella, 1989.

Continue

SOUNDING : CPT-07

DEPTH (m)	DEPTH (ft)	TIP RESISTANCE (tsf)	FRICTION RATIO (%)	SOIL BEHAVIOR TYPE	N(60)	N1(60)	Dr (%)	Su (tsf)	PHI (Degrees)
21.300	69.88	299.79	3.12	*SAND to CLAYEY SAND	100	99			
21.450	70.37	349.52	1.99	SAND to SILTY SAND	87	57	91		42.5
21.600	70.87	306.10	3.09	*SAND to CLAYEY SAND	100	100			
21.750	71.36	324.88	1.67	SAND to SILTY SAND	81	53	89		42.0
21.900	71.85	292.12	1.69	SAND to SILTY SAND	73	48	85		41.5
22.050	72.34	346.21	1.55	SAND to SILTY SAND	87	56	90		42.5
22.200	72.83	390.76	1.56	SAND to SILTY SAND	98	63	94		43.0
22.350	73.33	213.64	1.38	SAND to SILTY SAND	53	35	76		39.5

TIP RESISTANCE CORRECTED FOR END AREA EFFECT
*INDICATES OVERCONSOLIDATED OR CEMENTED MATERIAL
ASSUMED TOTAL UNIT WT = 115 pcf
ASSUMED DEPTH OF WATER TABLE = 15.0 ft
N(60) = EQUIVALENT SPT VALUE (60% Energy)
N1(60) = OVERBURDEN NORMALIZED EQUIVALENT SPT VALUE (60% Energy)
Dr = OVERBURDEN NORMALIZED EQUIVALENT RELATIVE DENSITY
Su = OVERBURDEN NORMALIZED UNDRAINED SHEAR STRENGTH
PHI = OVERBURDEN NORMALIZED EQUIVALENT FRICTION ANGLE

HOLGUIN, FAHAN & ASSOCIATES, INC.

Interpretations based on: Robertson and Campanella, 1989.